

STUDY ON THE MECHANISM OF EARTHQUAKE-INDUCED LANDSLIDE FOCUSING ON THE PULSE-LIKE GROUND MOTIONS

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**STUDY ON THE MECHANISM OF EARTHQUAKE-
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LIKE GROUND MOTIONS**

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**STUDY ON THE MECHANISM OF EARTHQUAKE-
INDUCED LANDSLIDE FOCUSING ON THE PULSE-LIKE
GROUND MOTIONS**

A Thesis Submitted
In Partial Fulfillment of the Requirements
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By
Mingyao Xia



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GRADUATE SCHOOL OF ENGINEERING
KYUSHU UNIVERSITY

Fukuoka, Japan

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CERTIFICATE

The undersigned hereby certify that they have read and recommended to the Graduate School of Engineering for the acceptance of this thesis entitled, “***Study on the mechanism of earthquake-induced landslide focusing on pulse-like ground motions***” by **Mingyao Xia** in partial fulfillment of the requirements for the degree of **Doctor of Engineering**.

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ABSTRACT

Earthquake-induced landslide is one of the significant secondary hazards of an earthquake. Even in a country like Japan, one of the advanced countries in earthquake disaster prevention, earthquake-induced landslides could still cause catastrophic disasters. For example, the 2016 Kumamoto earthquake (M7.3) and the 2018 Hokkaido Eastern Iburi earthquake (M6.5) induced landslides resulting in 15 victims and 39 victims separately, even though no victim resulted from building collapse due to the advance of quake-resistance technology of buildings. Thus, it is necessary and important to develop a more accurate prediction method on landslide-prone slopes based on the active fault information and potential earthquake magnitude. However, the mechanism of the earthquake-induced landslide has not been completely clarified up to now.

Slope stability is related to both the slope's inherent factors such as geological and physical properties and the external triggering forces from seismic waves. Up to now, the peak ground acceleration (PGA) of the seismic wave has been considered as one of earthquake triggering factors since it represents the maximum force of a seismic wave. Thus, PGA is proportional to slope instability in some slope stability analysis methods such as the Pseudo-static analysis and landslide hazard mapping. However, it is difficult to explain the following two phenomena occurred during some earthquakes: (1) large-scale landslides occurred in the area with small PGA while no landslide in the area with large PGA; (2) one side slopes collapsed while the opposite side slopes back to back remain stable even though PGA is the same for the two back to back slopes. In fact, it is believed that slope stability should also be affected by the energy of the seismic wave. It is found that PGA is not always proportional to seismic energy which is related to not only the magnitude but also the frequency of a seismic wave. Therefore, how to clarify the mechanism of earthquake-induced landslides from the view of seismic energy is very important in order to explain the above two phenomena.

This study aims to (1) clarify the mechanism of earthquake-induced landslides focusing on the pulse-like ground motion (PLGM) by using Discontinuous Deformation Analysis (DDA) simulation and (2) explain the above two phenomena. PLGM contains large seismic energy whatever PGA is in a seismic wave. DDA is a powerful numerical method for landslide simulation considering the dynamic behaviours of a rock mass system involving discontinuities. At first, the original DDA is improved so as to be used in slope stability analysis under seismic waves by modifying vertex-to-vertex contact processing and adding a unified tensile fracture criterion. Then, a method for evaluating the dynamic failure of the slope considering seismic energy is proposed in DDA. It is shown that a slope can collapse under a PLGM seismic wave with small PGA while it keeps stable under a large PGA seismic wave without PLGM. Thirdly, a case study on a landslide in the 2016 Kumamoto earthquake triggered by PLGM with a small PGA is performed to verify the proposed method. Finally, a possible mechanism of earthquake-induced landslides based on PLGM is clarified to explain the above mentioned two phenomena by investigating PLGM and PGA characteristics of various seismic waves and slope's physical properties.

The contents of the thesis are organized as follows:

Chapter 1 introduces the background, scope and objectives of the research. Previous studies on the mechanism of landslides, analytic methods of earthquake-induced landslides, and studies on PLGM in the architecture field are reviewed and existing problems are briefly discussed.

Chapter 2 reviews the state-of-the-art of 2D DDA and numerical simulations for earthquake-induced landslides. The basic theory of DDA formulation and developments of the DDA program are introduced. The advantages of DDA application to numerical landslide simulation are reviewed. The problems of the original DDA are also clarified for improvement and extension in this study.

Chapter 3 improves DDA by modifying the vertex-to-vertex contact processing and incorporating a unified tensile fracture criterion for better slope stability analysis under seismic conditions. At first, a missing case of vertex-to-vertex contacts is found in the original DDA, which has no significant effect on

general applications but the effect cannot be ignored on the landslide simulations under seismic waves. Thus, a new concept for vertex-to-vertex contact is introduced to enhance the contact detection algorithm. Then, a unified fracture criterion by combining shearing strength with tensile strength is incorporated into DDA to simulate the rock fracture behavior of various materials since only the Mohr-Coulomb criterion and maximum normal stress criterion are adopted in the original DDA, which can not be used to analyze the materials with correlation between shearing and tensile strengths. The accuracy of the improved DDA is validated by numerically reproducing several Brazil disc split tests, which are well used for verifying numerical methods. The simulated results agree well with those from experiment tests while no good results can be obtained by using the original DDA.

Chapter 4 proposes a method for slope stability analysis under earthquakes by considering the energy of seismic waves in DDA simulation. Earthquake-induced slope failure is related to not only PGA representing the maximum inertial force of a seismic wave but also seismic energy reflecting both amplitudes and frequencies. Although many PGA-based analytical methods have been developed, there is no energy-based slope stability analysis. For this reason, a dynamic failure method considering the effect of fatigue and cumulative damage of a seismic wave is proposed to analyze earthquake-induced slope failure by extending DDA as follows. 1) An effective-repeated loading (ERL) stress is defined based on slope's physically static strengths; 2) The cyclic number N of ERL accumulates as loading stress is larger than ERL stress in DDA simulation for the whole seismic wave. 3) Once the N reaches the critical number N_c , the slope failure occurs. A practical slope is modelled under two cases of seismic waves: large energy but small PGA (Case A: PLGM seismic wave) and large PGA but small energy (Case B: Non-PLGM seismic wave). The results show that the slope gets failure by using the energy-based DDA while keeps stable by using the original PGA-based DDA for Case A; but the results are inversed for Case B, which are also in agreement with seismic response analysis of SDOF system. Thus, the accuracy and effectiveness of the proposed energy-based DDA are validated and the energy of seismic waves is a key triggering factor in slope stability analysis under earthquakes.

Chapter 5 clarifies the mechanism of earthquake-induced landslides focusing on PLGM by explaining the two phenomena. A PLGM seismic wave contains a large velocity pulse waveform which is of large seismic energy. PLGM is usually caused by the so-called forward directivity effect in the near-fault area. PLGM seismic waves are identified at 3 K-NET observation sites in the Aso area where large-scale landslides occurred. Non-PLGM seismic waves with $PGA > 600$ gal are also recorded at other 2 K-NET observation sites in the around areas where no landslide occurs. A DDA slope model is analyzed by using the energy-based method under the all 5 seismic waves. The results show that the slope gets failure under the 3 PLGM waves with small PGA but keeps stable under the 2 Non-PLGM waves with larger PGA. Thus, PLGM is one of key factors of triggering landslides in the near-fault area. In order to explain the second phenomenon, a symmetrical DDA model of two back-to-back slopes is analyzed. The results show that one slope can get failure while the other side slope keeps stable. The aspect of the failure slope is the same as direction of the PLGM velocity pulse but it is inversed for the case of shear failure dominated. Therefore, it is shown that the seismic energy from PLGM is the major triggering factor of earthquake-induced landslides in near-fault areas. It should be paid more attention to PLGM in the possible forward directivity effect area near seismic fault in predicting landslide-prone slopes.

Chapter 6 summarizes the conclusions of the study and proposes recommendations for future research.

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LIST OF NOTATIONS OR ACRONYM

Notation or Acronym	Description
DDA	Discontinuous Deformation Analysis
PLGM	Pulse-like ground motion
PGA	Peak ground acceleration
GSI	Geospatial Information Authority of Japan
USGS	United States Geological Survey
PGV	Peak ground velocity
CSV	Cumulative squared velocity
t	Time
V, v	Velocity
F_S	Factor of safety
W	Weight
α	Slope angle
ϕ, φ	Friction angle
k	Pseudo-static coefficient
a_h	Horizontal pseudo-static acceleration
g	Gravitational acceleration
a_c	Critical acceleration
2D DDA	Two-dimensional Discontinuous Deformation Analysis
FDM	Finite difference method
BEM	Boundary element method
DEM	Discrete element method
3D DDA	Three-dimensional DDA

Notation or Acronym	Description
D_i	Deformation matrix
U	Total displacement of a point
T_i	Displacement transformation matrix
D	Displacement matrix
$\dot{D}$	Velocity matrices
$\ddot{D}$	Acceleration matrices
M	Mass matrix
C	Damping matrix
K	Stiffness matrix
F	Force vector
u_0, v_0	Rigid body translations in x -, y - directions
r_x, r_y	Rotations around x -, y - directions
$\varepsilon_x, \varepsilon_y$	Normal strains in x -, y - directions
γ_{xy}	Shear stains in x -, y - directions
n	Step number
t_0	Rotations around z - directions
Δt	Step time duration
OCI	Open-close iteration
Π	Potential energy
Ω	Domain of integration
$\vec{f}$	Force vector
$\vec{s}$	Displacement vector
i, j	Indexes
(x_i, y_i)	Coordinates of point i
J	Jacobi determinant

Notation or Acronym	Description
S	Simplex integral
T	Displacement transformation matrix
d_{ji}	J -th displacement component of block i
ϵ	Strain vector
E	Young's modulus
ν	Poisson's ratio
σ	Stress vector
E-E	Crossing edge-edge contact
V-V	Vertex-vertex contact
V-E	Vertex-edge contact
σ_0	Initial stress
f_x, f_y	Body force in x -, and y - directions
ρ	Mass per unit volume
$k_n, k_s(k_\tau)$	Normal and shear contact spring stiffness
$\mathbf{n}, \boldsymbol{\tau}$	Normal and parallel vector to the entrance point
$\lambda_{b1}, \lambda_{b2}$	Auxiliary parameters
d_n	Overlapping distance
V	Volume
A_0	Double of the area
$\mathbf{v}_1, \mathbf{v}_2$	Auxiliary vectors
L_{rs}	Algebraic complement
$\mathbf{H}, \mathbf{G}$	Coefficient matrices
τ_c	Cohesion
σ_c	Tensile stress
A	Area

Notation or Acronym	Description
O-DDA	Original DDA
I-DDA	Improved DDA
d_0	The threshold value of distance
d_v	Distance between two vertices
R	The maximum displacement ratio
N	Total number of computation steps
d_s	The relative shear displacement
l	Length of the contact
T	Tensile strength
σ	Normal stress
τ	Shear stress
a	The ratio of normal stress to shear stress
θ_f	Fracture angle
P	Loading force
σ_t	The splitting tensile strength
σ_n	The peak value of measured normal stress
N_f	Number of loading cycles at the time of failure
τ_{f_Nf}	Shear strength at the time of failure after loading N_f
$\tau_{f_N=1}$	Shear strength under monotonic loading
f_1	Cyclic fatigue function
f_2	Cyclic damage function
d	Slope of the initial and failure points
τ_{d_Nf}	Cyclic residual shear strength exerted after N cycles
R_{res}	Residual shear strength ratio
SDOF	Single-degree-of-freedom

Notation or Acronym	Description
S_a	Spectral acceleration
I_A	Arias intensity
EPA	Effective peak acceleration
EPV	Effective peak velocity
E_r	Cumulation energy ratio
E_p	Pulse-like energy ratio
v_{pulse}	Velocity of extracted pulse ground motion
$v_{original}$	Velocity of original ground motion
B_1	Base slope block
M_1	Mass of base slope block
M_0	Mass of the potential sliding blocks
a_s	Input acceleration from seismic wave
DOP	Direction of the pulse
k_{e1}, k_{e2}	Coefficients of effective-repeated loading stress
ERLS	Effective-repeated loading stress
ERL	Effective-repeated loading
N_c	The critical number of effective-repeated loading

CHAPTER 1

INTRODUCTION

1.1. BACKGROUND

Earthquake-induced landslide is one of the most destructive natural hazards. Damages caused by earthquake-induced landslides are sometimes even larger than damages from the shaking of the earthquake itself because of the development of structure engineering ([Causes L., 2001](#); [Hideaki Marui, 2015](#)). On the one hand, an earthquake-induced landslide usually has a long runout and high velocity, which can cause significant and even catastrophic damage to infrastructure and people's lives. On the other hand, landslides can further cause a geological disaster chain by forming landslide dams that block valleys and stream channels. This can further cause dammed lake if the dam fails, subsequent downstream flooding, or debris flow disaster.

Scientific studies on earthquake-induced landslides have a long history ([Garwood et al., 1979](#); [Keefer, D. K., 2002](#); [Harp and Jibson, 1996](#); [Malamud et al., 2004](#); [Parise and Jibson, 2000](#); [Sassa et al., 1996, 2005](#)), which can be dated back to a hundred years ago since the 1906 San Francisco earthquake. [Varnes, D. J. \(1958\)](#) firstly defined the term "landslide" to denote a wide variety of processes that result in the downward and outward movement of slope-forming materials composed of natural rock, soils, artificial fills, or combinations of these materials. [Keefer, D. K. \(1984\)](#) claimed that the term "landslide" encompasses many phenomena involving

the lateral and downslope movement of earth materials. Causes L. (2001) summarized that the materials may move by falling, toppling, sliding, spreading, or flowing, and the various types of landslides can be differentiated by the kinds of material involved and the mode of movement. According to Varnes' classification of slope movements, a classification system for landslide types based on the types of slope movements and materials was proposed by Causes L. (2001), as shown in Table 1.1.

Table 1.1 Types of landslides (Causes L, 2001)

Type of Movement		Type of Material	
		Bedrock	Engineering Soils
			Predominantly Coarse Predominantly Fine
Falls		Rock fall	Debris fall Earth fall
Topples		Rock topple	Debris slide Earth slide
Slides	Rotational	Rock slide	Debris slide Earth slide
	Translational		
Lateral spreads		Rock spread	Debris spread Earth spread
Flows		Rock flow (deep creep)	Debris flow (soil creep) Earth flow (soil creep)
Complex		Combination of two or more principal types of movements	

The research on earthquake-induced landslides already has long history, but the failure mechanic of earthquake-induced landslide has not been clearly clarified. Many large-scale were triggered in each catastrophic earthquake event, which caused many deaths and huge losses. Up to now in many developed countries, earthquake-induced landslide is still one of the most dangerous geological hazards.

1.1.1. SOME UNABLE EXPLAINED LANDSLIDE PHENOMENA

Recently, earthquake-induced landslides have frequently occurred in world (Yin et al., 2009; Fan et al., 2018; Wang et al., 2009; Song et al., 2019; Wang et al., 2019). Even in the country like Japan, one of the advanced countries in earthquake disaster prevention, earthquake-induced landslides caused serious damage to infrastructure and lifelines and led to miserable fatalities. For example, the 2016 Kumamoto earthquake (M7.3) induced landslides resulting in fifteen of the victims (Chen et al., 2019; Shinoda et al., 2019). The 2018 Hokkaido Eastern Iburi earthquake (M6.5) induced landslides resulting in 39 of the victims but no victim resulted from building collapse due to the advance of quake-resistance technology of buildings (Yamagishi & Yamazaki, 2018; Osanai et al., 2019). Thus, it is necessary and important to develop a more accurate prediction method on landslide initiation based on the active fault information and potential earthquake magnitude. However, the mechanism of the earthquake-induced landslide has not been clearly clarified up to now. There are two phenomena confusing landslide researchers.

1.1.1.1. LARGE-SCALE LANDSLIDES OCCURRED IN THE AREA WITH SMALL PGA BUT NO LANDSLIDE IN THE AREA WITH LARGE PGA

A strong earthquake (M7.3) occurred in Kumamoto prefecture, Japan in 2016, called the Kumamoto earthquake. It induced four large-scale landslides which caused ten fatalities (Figure 1.1) in the Aso area where there are a great number of seismic observation stations of Kyoshin Network (K-NET), which is a network of strong-motion seismographs installed at approximately 1,000 locations nationwide in Japan.

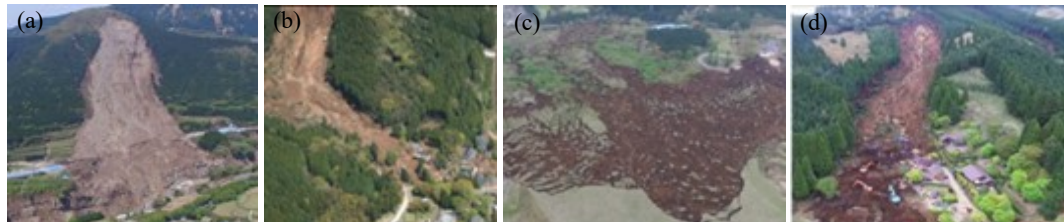


Figure 1.1 Four large-scale landslides: (a) the Aso Ohashi landslide, (b) the Tachino landslide, (c) the Takanodai landslide, (d) the Hinotori landslide. (Edited

from the 2016 Kumamoto earthquake data profile in homepage of Kumamoto Prefecture, <https://www.pref.kumamoto.jp/uploaded/attachment/24488.pdf>)

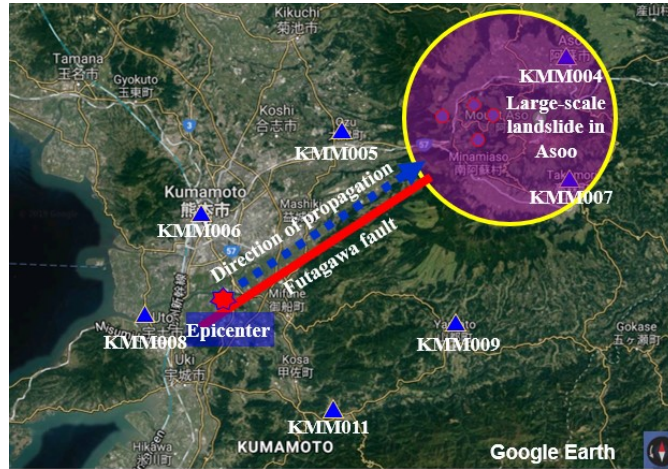


Figure 1.2 The locations of the four landslides (circle points), epicenter (the pentagram), the seismic fault (red line) and observation points of K-NET (triangle) (Edited from Google map)

Table 1.2 Seismic ground motions in near-fault region in the 2016 Kumamoto earthquake

Site	PGA-EW (gal)	PGA-NS (gal)	PGV (cm/s)
KMM004	347	261	84
KMM005	482	526	52
KMM007	420	279	37
KMM006	616	827	52
KMM008	771	651	86
KMM009	640	777	39
KMM011	602	598	36

The seismic waves from the earthquake struck area have been analyzed, as shown in Figure 1.2. It has been found from Table 1.2 that large-scale landslides occurred in the area with small PGA but no landslide in the area with large PGA. The epicenter was located in the Futagawa fault zone with a length of 20 km and a width of 12.5 km in the Kumamoto district (Figure 1.2). The fault strike is 235°,

the fault dip is 60° , slip dislocation is about 4.1 m. The earthquake struck area mainly are mountainous areas with an elevation of 200 m or more, along the Futagawa fault zone in the northeast-southwest (NE-SW) direction. The hills and plateaus in those mountainous areas consist mainly of pyroclastic flow deposits and lava associated with the formation of the Aso caldera (Shinoda et., 2019). The soil characteristics of the slopes should be similar in those stations with different PGA. Why large-scale landslides occurred in the area with smaller PGA and no landslide occurred in the area with larger PGA remains an unresolved issue.



Figure 1.3 Ground surface in northern Atsuma Town after 2018 earthquake (From Geospatial information authority of Japan)

Meanwhile, the 2018 Hokkaido Eastern Iburi earthquake (M6.7), called the Hokkaido earthquake, induced a large number of landslides in Japan (Figure 1.3). Forty-one fatalities were caused by the landslides although no fatality was caused by building collapse because of the advanced technology of earthquake-resistant construction in Japan. The locations of major landslides, the epicenter, the seismic fault and the four seismic observation stations are also shown in Figure 1.4 (a). The epicenter depth is about 33 km. The seismic fault is a reverse slip fault with a length of 14km and a width of 15.9 km. The fault strike is 358° , the fault dip angle is 74° , slip dislocation is about 1.3 m. Figure 1.4 (b) shows a near-real-time spatial estimate of earthquake-triggered landslide hazard map for Hokkaido HEIE landslides conducted by United States Geological Survey (USGS). The actual landslide distribution is significantly different from the post estimation of landslide distribution. The post estimation of landslide distribution is centered on the larger PGA areas during the earthquake, while the actual landslide areas are located close to the other monitoring site HKD128 where the PGA is not highest.

Table 1.3 Seismic ground motions in near-fault region in the 2016 Kumamoto earthquake

Site	PGA-EW (gal)	PGA-NS (gal)	PGV (cm/s)
HKD128	673	628	105
HKD126	661	388	165
HKD125	768	573	54
HKD104	349	276	15

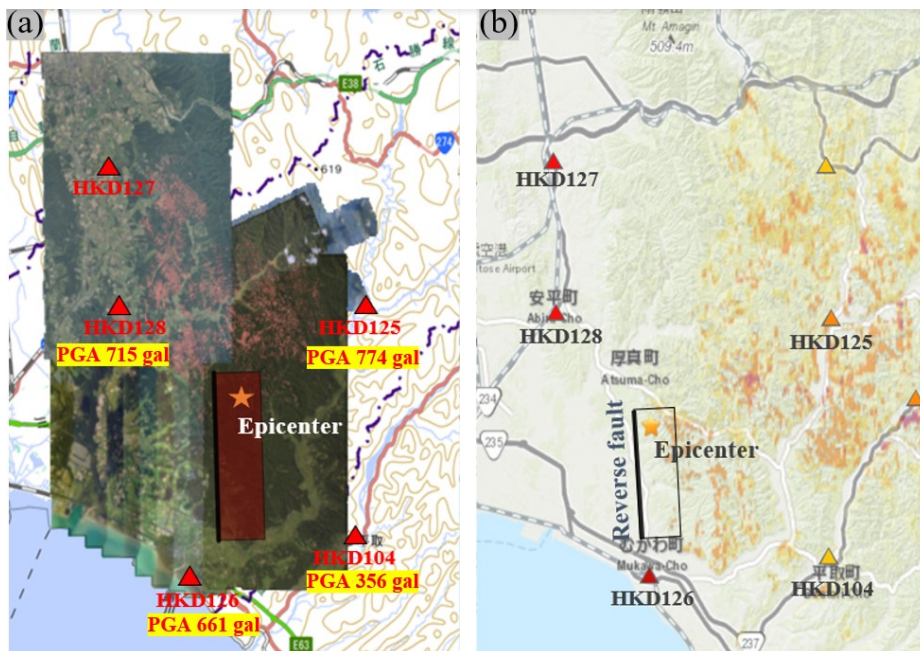


Figure 1.4 The 2018 Hokkaido eastern Iwuri earthquake: (a) Real landslide distribution (Edited from Geospatial Information Authority of Japan, GSI) (b) Estimation of earthquake-induced landslide hazard map from USGS

1.1.1.2. LANDSLIDE OCCURRING ON THE SLOPES WITH A SPECIFIC ORIENTATION

The other confusing phenomena occurred in the 2018 Hokkaido Eastern Iwuri earthquake is that most west-faced slopes collapsed while the east-faced slopes back to back remain stable. Figure 1.5 shows the landslide distribution near the Atsuma

town which is the main earthquake-induced landslide area in the Hokkaido earthquake. The red lines in this figure are ridgelines, the black lines (AA' and BB') are the cross-section lines. Figure 1.5 (c) and (d) show the elevations of cross-sections of the back to back slopes along lines AA' and BB' in Figure 1.5 (a) and (b), which can be obtained from map of the 2018 Hokkaido landslide distribution provided by GSI. It can be seen that the landslides are mostly triggered at the west side even though the west-faced slopes are gentler than east-faced slopes. Since PGA should be the same for two slopes back to back, why one collapsed while the other remains stable cannot be explained by the existing mechanism.

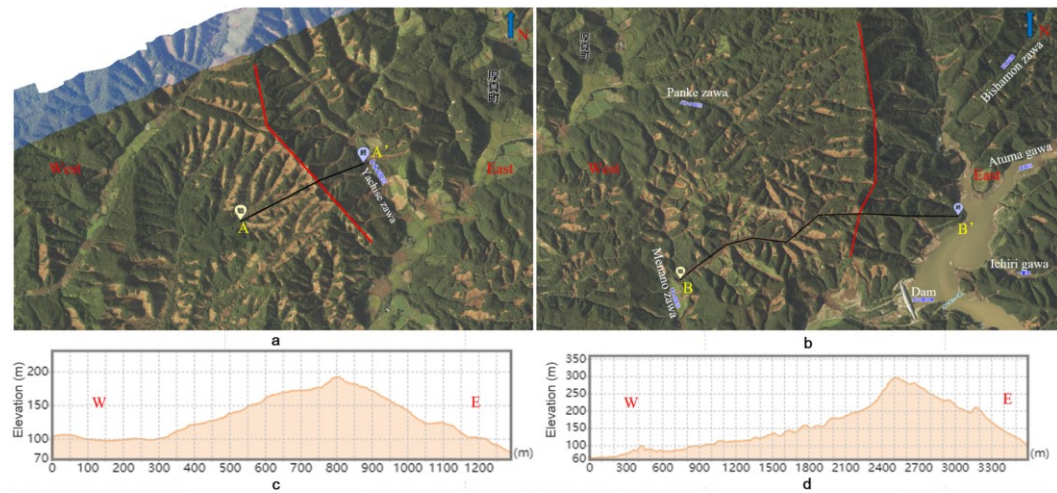


Figure 1.5 West-faced slopes collapsed while east-faced slopes remain stable
(Edited from the 2018 Hokkaido landslide hazard map of GSI)

1.1.2. LANDSLIDE INITIATION AND EARTHQUAKE INTENSITY PARAMETERS

Landslide initiation is not only related to the slope inherent factors such as slope shape and geological and physical properties and discontinuities, but also affected by the external triggering factors such as earthquake ground motions, rainfall, and human activities, as shown in Figure 1.6. Both the external triggers and inherent factors should be considered in estimating the potentially dangerous slopes under earthquakes. For earthquake-induced landslides, the earthquake triggering force is the most important factor that should be considered for the initiation of landslides. Many research used peak ground acceleration (PGA) as one of the important parameters to study the initiation of co-seismic landslides (Harp and

Wilson, 1995; Jibson et al., 2000; Liao, 2000; Khazai and Sitar, 2004; Wang et al., 2010; Yuan et al., 2013). Liao (2000) believed that earthquake-induced landslides are mainly located in regions with a $\text{PGA} > 2.5 \text{ m/s}^2$ based on his study on the Jiji (Chi-Chi) earthquake (M7.6). Wang et al. (2010) reported that the threshold of the PGA is about 0.7 m/s^2 by studying the relationships between ground motion parameters and earthquake-induced landslides in the Wenchuan earthquake. while according to the distribution of landslides induced by the 2008 Mw 7.9 Wenchuan earthquake, Yuan et al. (2013) obtained the Thresholds of critical PGA for landslide failure are 0.21g for average horizontal PGA. Studies showed that the critical PGA values triggering landslides change from earthquake to earthquake. It is common that the instantaneously peak amplitude of seismic wave (e.g., PGA) is applied in seismic slope stability analysis, but some researches also propose that the whole energy of seismic wave should be considered (Mollaioli and Bosi, 2012; Zhai et al., 2013, 2018; Chang et al., 2016; Kardoutsou et al., 2017). There is no doubt that the slope may collapses if both PGA and energy of seismic wave are large and remains stable if PGA and energy are small. While in some special cases, slope collapses when the PGA is small but the energy is large and remains stable when the PGA is large but the energy is small. In those cases, it is difficult to study the initiation of co-seismic landslide exclusively with the critical value of PGA. As as introduced previously in section 1.1.1, some phenomena on landslide initiation in recent earthquake events cannot be clearly clarified by PGA and a more accurate method is necessary for seismic slope stability analysis.

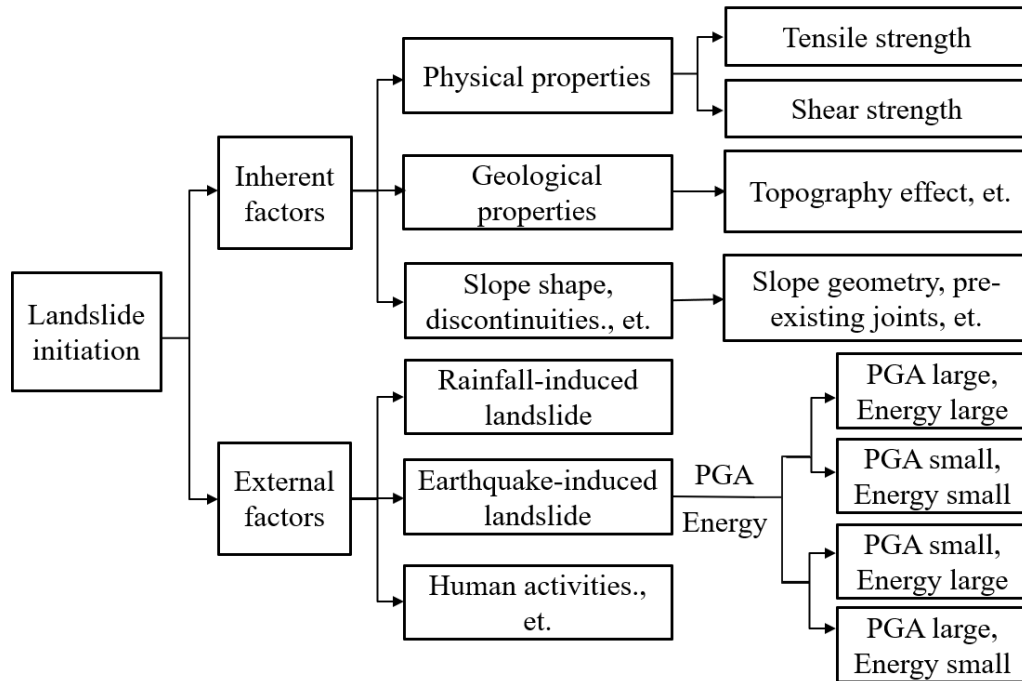


Figure 1.6 The main factors for landslide initiation

Many researchers have also paid close attention to other intensity parameters responsible for the damage potential of ground motions including peak ground velocity (PGV), spectral acceleration and Arias intensity (Malhotra, 1999; Liao et al., 2001; Mavroeidis et al., 2004; Akkar et al., 2005; Tothong and Cornell, 2008; Yang et al., 2009). Most parameters are not related well with the initiation of earthquake-induced landslides and it is difficult to clarify the landslide initiation by use of those intensity parameters.

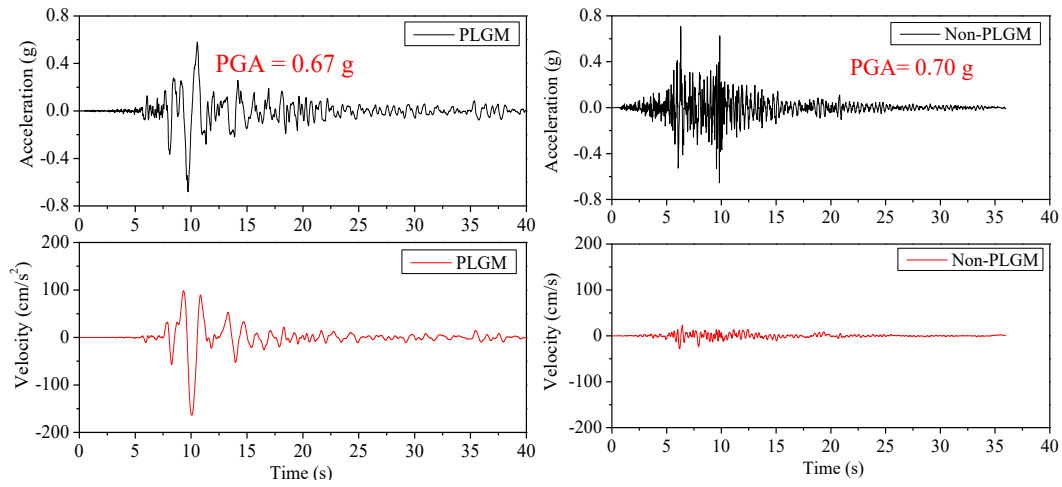


Figure 1.7 Examples of PLGM and Non-PLGM

Pulse-like ground motion (PLGM) is a kind of damaged seismic wave with a strong velocity pulse, which has been of special concern for earthquake engineering for a long time (Mavroeidis and Papageorgiou, 2003; Stewart et al., 2003; Baker, 2007; Shahi and Baker, 2013; Zhai et al., 2013; Chang et al., 2016; Zhao et al., 2021; Daei et al., 2021). Figure 1.7 shows the acceleration and velocity waves of PLGM and Non-PLGM. There is a significant difference between PLGM and Non-PLGM that PLGM has a strong velocity pulse while no obvious velocity pulse can be observed from Non-PLGM, even though both waves have the approximately equal values of PGA. Since PLGM has a strong velocity pulse which contains large kinetic energy, PLGM is considered to be strongly related to structural damage and can cause larger damage on structures than non-PLGM. However, PLGM has been studied extensively in the architecture field while it remains unclear in co-seismic slope stability analysis. The PLGMs might be a key factor triggering landslides, which deserves further study. Most previous studies on earthquake-induced landslides have been concentrated on the PGA of seismic record, whether the PLGM characteristic has an influence on the initiation of landslides has been

neglected all the time. The relationship between the initiation of earthquake-induced landslides and PLGMs remains unclear.

1.1.3. PULSE-LIKE GROUND MOTIONS

The research on pulse-like ground motion can date back to the 1966 Parkfield and the 1971 San Fernando earthquake when a ground motion with great velocity pluses was observed on near-fault region causing severe damage to structure (Mavroeidis and Papageorgiou, 2003). Recently, many researchers have proposed the quantitative classification method of near-fault pulse-like ground motion, which provides a quantitative method for determining the pulse-like ground motions (Baker, 2007; Shahi and Baker, 2013; Zhai et al., 2013). Good knowledge of this classification method can be useful for the representation of PLGM on landslides triggering. Baker (2007) developed a criterion for classifying the near-fault PLGM in the fault-normal direction. Then Shahi and Baker (2013) improved the algorithm for the identification of PLGM on multiple orientations based on Baker's method. Other researchers also proposed many methods for the classification of PLGM (Mavroeidis and Papageorgiou, 2003; Zhai et al., 2013). Based on those methods, PLGM can be quantitatively identified and extracted. Those researches provide the possibility to quantitatively analyze the earthquake-induced landslides focusing PLGMs. The cumulative squared velocity (CSV) is usually calculated to represent the energy of PLGM by the following formula (Baker, 2007; Zhai et al., 2013; Shahi and Baker, 2014; Chang et al., 2016):

$$CSV(t) = \int_0^t V^2(t) dt \quad (1.1)$$

where $CSV(t)$ denotes the cumulative squared velocity at time t and $V(t)$ is the ground-motion velocity at time t .

Not all seismic waves contain pulse-like waves since PLGM occurs only in special geological conditions. Previous researches on PLGM indicated that the forward directivity effect is one of the main causes of PLGM is in the near-fault region. It is also known as the Doppler effect (Somerville, 2003), as shown in Figure 1.8, point 0 is assumed to be the initial rupture location. As rupture propagates from

point 0 to 3, the seismic wave caused by rupture propagation is continuously catching up with the seismic waves generated by previous rupture points. In the forward direction of rupture propagation, velocity waves can be gradually accumulated into a strong velocity pulse.

For strike-slip and dip-slip faults, when the direction of fault slip is consistent with the site and the propagation velocity of rupture nearly equal the propagation velocity of the shear waves, PLGM is most likely to occur because of the forward directivity effect. Additionally, the site location and the fault geometry may also produce velocity pulses at the site, but the forward directivity effect is presumed to be the main cause of many near-fault pulse-like ground motions. Shahi and Baker's (2013) method can be used as a quantitative algorithm for the identification of PLGM caused by the forward directivity effect.

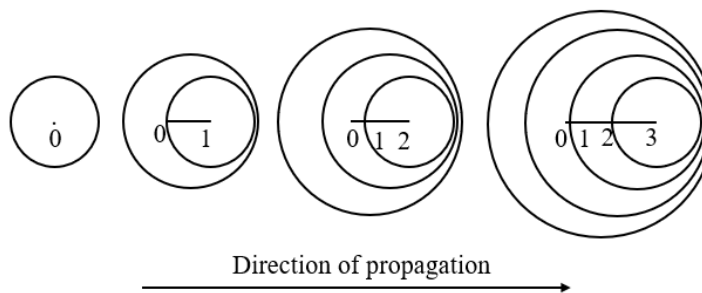


Figure 1.8 Schematic explanation of the forward-directivity effect (Somerville, 2003)

1.2. APPROACHES FOR EARTHQUAKE-INDUCED LANDSLIDES

Over the past decades, many methods have been proposed to analyze earthquake-induced landslides. In general, the methods developed to assess the stability of slopes during earthquakes can be divided into three categories: (1) pseudo-static methods, (2) Newmark's permanent-displacement methods, and (3) stress-strain methods (Jibson, 2011).

1.2.1. PSEUDO-STATIC ANALYSIS

Terzaghi (1950) first presented the pseudo-static method, in which the horizontal ground acceleration is applied to simply model the seismic shaking as a permanent body force in the static limit-equilibrium analysis. The safe factor of the slopes under earthquake can be obtained based on the limit-equilibrium theory. Figure 1.9 shows a planar slip surface in a slope consisting of dry, cohesionless material. The pseudo-static factor-of-safety of a slope under earthquake can be expressed as

$$F_s = \frac{[W \cos \alpha - kW \sin \alpha] \tan \phi}{W \sin \alpha + k W \cos \alpha} \quad (1.2)$$

where F_s is the pseudo-static factor of safety, W is the weight per unit length of slope, α is the slope angle, ϕ is the friction angle of the slope material, and k is the pseudo-static coefficient, calculated by

$$k = \frac{a_h}{g} \quad (1.3)$$

where a_h is the horizontal pseudo-static acceleration and g is gravitational acceleration.

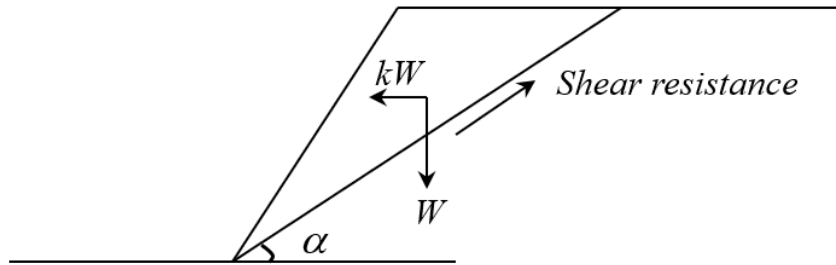


Figure 1.9 Force diagram of a landslide in cohesionless soil that has a planar slip surface. (Jibson, 2011).

The pseudo-static method is oversimplified method that has obvious drawbacks for evaluating of seismic stability of a slope. The selection of the pseudo-static coefficient has significant influence on the accuracy of the slope stability analysis. It is important to select the appreciate pseudo-static coefficient for modeling the seismic shaking. However, it only assumes that the earthquake force is constant and acts only in a direction in the earliest method. To solve this problem,

more studies have been conducted to analyze the selection of the pseudo-static coefficient (Seed, 1979; Kramer, 1996; Bray and Rathje, 1998; Stewart et al., 1998; Bray and Travarasrou, 2009). Seed (1979) summarized that the horizontal seismic coefficients range from 0.1 to 0.15 by analyzing the factor of safety value for 14 large dams worldwide in the published reports. Bray and Rathje (1998) related the seismic coefficient to the PGA of bedrock, ranging from 0.6 to 0.75PGA_{rock}. Stewart et al. (2003) developed a procedure to calculate the pseudo-static coefficient based on a function of maximum horizontal ground acceleration, earthquake magnitude, source distance, and two possible levels of allowable displacement. Bray and Travarasrou (2009) proposed a straightforward approach that calculates the pseudo-static coefficient as a function of allowable displacement, earthquake magnitude, and spectral acceleration. Table 1.4 shows the recommended pseudo-static coefficients from previously various studies (Jibson, 2011). In summary, pseudo-static method can be a simple method to identify the factor-of-safety of a slope under earthquake, but it only shows a slope to be either stable or unstable, but the more detailed consequences of instability cannot be judged (Jibson, 2011).

Table 1.4 Pseudo-static coefficients from previously various studies

Investigator	Recommended Pseudo-static coefficient (k)	Recommended factor of safety (F_s)	Calibration conditions
Terzhagi (1950)	0.1 (R-F = IX)	>1.0	Unspecified
	0.2 (R-F = X)		
	0.5 (R-F > X)		
Seed (1979)	0.10 (M=6.50)	>1.15	<1 m displacement in earth dams
	0.15 (M=8.25)		
Marcuson (1981)	0.33–0.50×PGA/g	>1.0	Unspecified
Hynes-Griffin and Franklin (1984)	0.50×PGA/g	>1.0	<1 m displacement in earth dams
California Division	0.15	>1.1	Unspecified; probably

of Mines and Geology (1997)			based on <1 m displacement in dams
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Note R-F is Rossi-Forel earthquake intensity scale, M is earthquake magnitude, PGA is peak ground acceleration, g is acceleration of gravity.

1.2.2. NEWMARK'S PERMANENT-DISPLACEMENT ANALYSIS

Newmark's permanent-displacement analysis method is an alternative approach to evaluate the seismic slope stability, in which the permanent displacement is calculated as an important index for slope stability analysis. Newmark (1965) first proposed the basic elements of a procedure for evaluating a landslide as a rigid block that slides on an inclined plane, as shown in Figure 1.10. In this method, a critical acceleration a_c , also known as yield acceleration, is firstly determined by a pseudo-static analysis. Then, the parts of the recorded earthquake acceleration that exceed the critical acceleration are integrated twice to obtain the cumulative displacement of the landslide block. The critical acceleration a_c can be calculated as:

$$a_c = (F_s - 1)g \sin \alpha \quad (1.4)$$

where a_c is in terms of g, the acceleration of gravity, F_s is the static factor of safety, and α is the angle from the horizontal of the sliding surface.

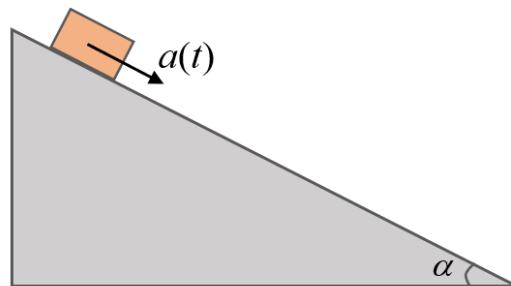


Figure 1.10 Newmark's sliding block model

The original Newmark's method is generally defined as a rigid sliding block analysis, in which the landslide mass is assumed as a rigid-plastic body with no internal deformation. This assumption indicates that the no permanent displacement at accelerations below the critical level and only deforms plastically when the critical acceleration is exceeded. For better application of Newmark's method in

landslide analysis, several extensions of Newmark's method have been proposed since the original Newmark rigid-block sliding method. Some research proposed that the landslide masses are not rigid bodies but deform internally when subjected to seismic shaking ([Seed and Martin, 1966](#); [Lin and Whitman, 1983](#)). Makdisi and Seed ([1978](#)) provided design charts that estimate sliding displacement with a function of critical acceleration, ground motion, and earthquake magnitude. Jibson ([2003](#)) allows specifying strain-dependent critical acceleration as well as bi-lateral displacement, and dynamic strength testing can be performed to determine dynamic shear strengths. Bray and Travasarou ([2007](#)) developed a simplified approach that used a coupled sliding-block model to produce a semiempirical relationship for predicting displacement. Up to now, permanent-displacement analysis has been applied in a variety of ways to seismic slope stability problems. But in the current status of the permanent-displacement analysis, the determination of the critical displacement leading to the failure of landslide is a difficult problem. Any level of critical displacement can be used according to the characteristics of the landslide material.

1.2.3. NUMERICAL STRESS-DEFORMATION ANALYSIS

Stress-deformation analysis, also known as numerical simulation method, has been well developed since the early twentieth century. In 1960, Ray Clough first developed the finite-element method of engineering analysis at the University of California, Berkeley ([Clough, 1960](#)). This method uses a mesh to model a deformable system, providing a valuable tool for modeling the static and dynamic deformation of soil systems. It soon began to be applied to slope stability analysis and be of great interest in this research. Subsequently, more and more numerical methods have been developed for stress-deformation analysis, including the finite-difference method (FDM) ([Mitchell and Griffiths, 1980](#)), boundary element method (BEM) ([Brebbia and Wrobel, 1980](#)), discrete element method (DEM) ([Cundall, 1971](#)) and discontinuous deformation analysis (DDA) ([Shi, 1989](#)). Stress-deformation analysis can provide the most realistic simulation of slope behavior under an earthquake. A lot of research on the earthquake-induced landslide events

are conducted by using those numerical method (Wu et al., 2011, 2017; Chen et al., 2018; Wang et al., 2019; Liu et al., 2019). Among them, DDA is one of the most popular methods for earthquake-induced landslides because it has a great advantage in simulating large displacement and rigid body movements of the discrete elements with arbitrary shapes, such as the fractured and jointed rock masses.

1.3. SCOPES AND OBJECTIVES

1.3.1. EXISTING PROBLEMS

As introduced in previous sections, the pseudo-static analysis was the earliest method, in which horizontal ground acceleration is applied to simply model the seismic shaking as a permanent body force in the static limit-equilibrium analysis. Newmark's permanent-displacement method is proposed to remedy the weaknesses of the overly simplistic pseudo-static analysis. However, methods of pseudo-static analysis and Newmark's method, focus on the effects of earthquake accelerations on the landslide mass, which only accounts for shear failure, but the role of tensile failure is not considered. With the advancement of computer techniques, numerical methods including finite-difference, distinct-element, and discrete-element modeling, are widely applied to earthquake-induced landslide analysis.

DDA is a powerful numerical method and shows a great advantage in simulating the dynamic behaviors of a rock mass system involving discontinuities. It has been well developed for static and dynamic landslide simulation since it was proposed. However, the seismic ground motion is a dynamic loading force with a long duration while the static shear strength is used for seismic conditions in numerical methods, the numerical method still has difficulties in describing the slope failure during an earthquake. In previous studies, the PGA is always believed to be the key triggering factor dominating the failure mechanic of the constitution model for earthquake-induced landslides. The larger the PGA is, the more dangerous the slope becomes. However, some phenomena of landslide initiation and evolution cannot be clarified by PGA. Many researchers have also paid close attention to other intensity parameters responsible for the damage potential of

ground motions including peak ground velocity (PGV), spectral acceleration, and Arias intensity. PLGM is of special concern in engineering. It has been reported that the large PLGM wave is strongly related to structural damage because the energy in the PLGM wave is much larger than non-PLGM. However, PLGM has been studied extensively in the architecture field while it remains unclear in co-seismic slope stability analysis. The failure mechanism of earthquake-induced landslides focusing on PLGM remains unclear. How to simulate dynamic behaviors of earthquake-induced landslides deserves further investigation.

1.3.2. OBJECTIVES OF THE THESIS

The main objective of this study is to investigate the mechanism of earthquake-induced landslides focusing on the PLGM by use of the 2D DDA method. DDA is a powerful numerical method and shows a great advantage in simulating the dynamic behaviors of a rock mass system involving discontinuities. To analyze earthquake-induced landslides focusing on PLGM, some improvements and extensions should be conducted at first for the 2D DDA method to achieve better calculation performance. Then, the mechanism of earthquake-induced landslides focusing on PLGM will be studied by using the improved DDA. In detail, this study can be concluded as those following steps:

- (1) Improvement of vertex-to-vertex contact processing and the unified tensile fracture criterion in 2D DDA for enhancing the calculation performance of block systems in seismic response analysis;
- (2) A method to evaluate slope failure subjected to PLGM by considering the energy of seismic waves;
- (3) Studies on the failure mechanism of earthquake-induced landslides focusing on the using DDA method.

1.4. FRAMEWORK OF THE THESIS

The thesis consists of the following chapters.

Chapter 1 introduces the background, scope and objectives of the research. Previous studies on the mechanism of landslides, analytic methods of earthquake-

induced landslides, and studies on PLGM in the architecture field are reviewed and existing problems are briefly discussed.

Chapter 2 reviews the state-of-the-art of 2D DDA and numerical simulations for earthquake-induced landslides. The basic theory of DDA formulation and developments of the DDA program are introduced. The advantages of DDA application to numerical landslide simulation are reviewed. The problems of the original DDA are also clarified for improvement and extension in this study.

Chapter 3 improves DDA by modifying the vertex-to-vertex contact processing and incorporating a unified tensile fracture criterion for better slope stability analysis under seismic conditions. At first, a missing case of vertex-to-vertex contacts is found in the original DDA, which has no significant effect on general applications but the effect cannot be ignored on the landslide simulations under seismic waves. Thus, a new concept for vertex-to-vertex contact is introduced to enhance the contact detection algorithm. Then, a unified fracture criterion by combining shearing strength with tensile strength is incorporated into DDA to simulate the rock fracture behavior of various materials since only the Mohr-Coulomb criterion and maximum normal stress criterion are adopted in the original DDA, which can not be used to analyze the materials with correlation between shearing and tensile strengths. The accuracy of the improved DDA is validated by numerically reproducing several Brazil disc split tests, which are well used for verifying numerical methods. The simulated results agree well with those from experiment tests while no good results can be obtained by using the original DDA.

Chapter 4 proposes a method for slope stability analysis under earthquakes by considering the energy of seismic waves in DDA simulation. Earthquake-induced slope failure is related to not only PGA representing the maximum inertial force of a seismic wave but also seismic energy reflecting both amplitudes and frequencies. Although many PGA-based analytical methods have been developed, there is no energy-based slope stability analysis. For this reason, a dynamic failure method considering the effect of fatigue and cumulative damage of a seismic wave is proposed to analyze earthquake-induced slope failure by extending DDA as follows. 1) An effective-repeated loading (ERL) stress is defined based on slope's

physically static strengths; 2) The cyclic number N of ERL accumulates as loading stress is larger than ERL stress in DDA simulation for the whole seismic wave. 3) Once the N reaches the critical number N_c , the slope failure occurs. A practical slope is modelled under two cases of seismic waves: large energy but small PGA (Case A: PLGM seismic wave) and large PGA but small energy (Case B: Non-PLGM seismic wave). The results show that the slope gets failure by using the energy-based DDA while keeps stable by using the original PGA-based DDA for Case A; but the results are inversed for Case B, which are also in agreement with seismic response analysis of SDOF system. Thus, the accuracy and effectiveness of the proposed energy-based DDA are validated and the energy of seismic waves is a key triggering factor in slope stability analysis under earthquakes.

Chapter 5 clarifies the mechanism of earthquake-induced landslides focusing on PLGM by explaining the two phenomena. A PLGM seismic wave contains a large velocity pulse waveform which is of large seismic energy. PLGM is usually caused by the so-called forward directivity effect in the near-fault area. PLGM seismic waves are identified at 3 K-NET observation sites in the Aso area where large-scale landslides occurred. Non-PLGM seismic waves with $\text{PGA} > 600 \text{ gal}$ are also recorded at other 2 K-NET observation sites in the around areas where no landslide occurs. A DDA slope model is analyzed by using the energy-based method under the all 5 seismic waves. The results show that the slope gets failure under the 3 PLGM waves with small PGA but keeps stable under the 2 Non-PLGM waves with larger PGA. Thus, PLGM is one of key factors of triggering landslides in the near-fault area. In order to explain the second phenomenon, a symmetrical DDA model of two back-to-back slopes is analyzed. The results show that one slope can get failure while the other side slope keeps stable. The aspect of the failure slope is the same as direction of the PLGM velocity pulse but it is inversed for the case of shear failure dominated. Therefore, it is shown that the seismic energy from PLGM is the major triggering factor of earthquake-induced landslides in near-fault areas. It should be paid more attention to PLGM in the possible forward directivity effect area near seismic fault in predicting landslide-prone slopes.

Chapter 6 summarizes the conclusions of the study and proposes recommendations for future research.

The thesis framework is shown in Figure 1.11.

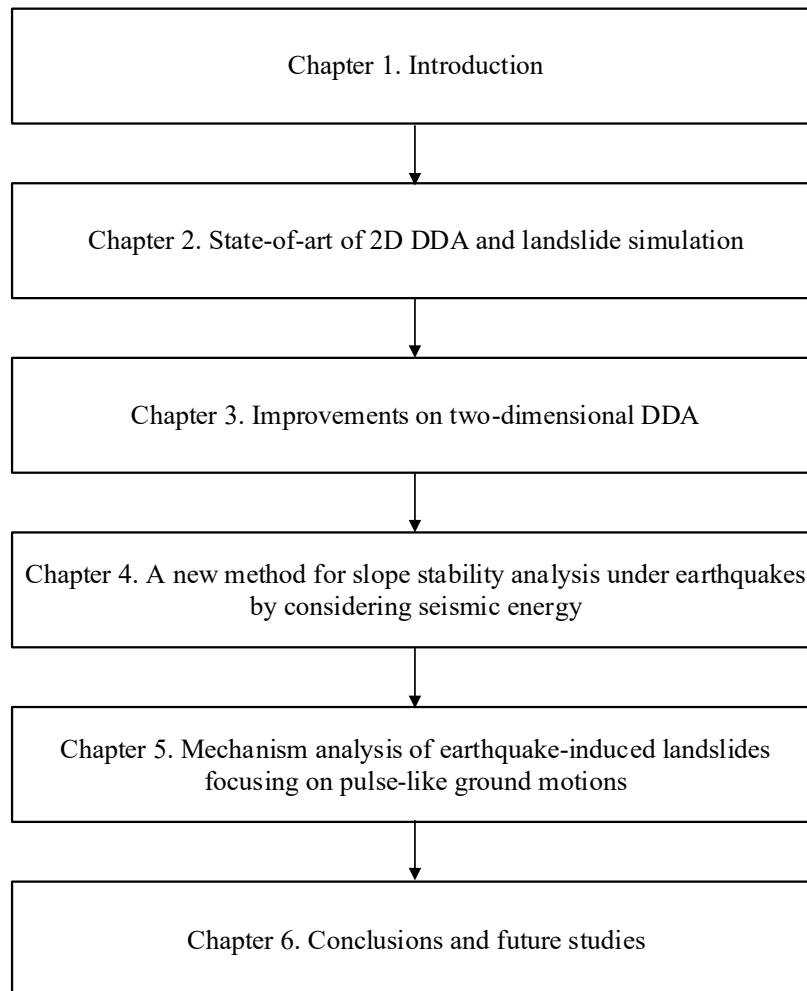


Figure 1.11 Framework of the thesis

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CHAPTER 2

STATE-OF-THE-ART OF 2D DDA AND LANDSLIDE SIMULATION

2.1. INTRODUCTION

Discontinuous deformation analysis (DDA), originally proposed by Dr. Shi (Shi, G. H. et al., 1989), is a useful tool for discontinuous media numerical simulation in geotechnical engineering and has garnered great research interest. Especially, the two-dimensional (2D) DDA method has developed significantly since its proposal (Hatzor, Y. H. et al., 2001; Yeung, M. R. et al., 2003; Tsesarsky, M. et al., 2005; Maclaughlin, M. M. et al., 2006; Wu, J. H., 2004, 2010, 2015; Yu, P. et al., 2019, 2020; Peng, X. et al., 2019a, 2019b, 2020), presenting a great advantage in simulating large displacement and rigid body movements. Many previous researches proved that DDA can effectively analyze the movement and deformation of blocks with arbitrary geometries numerically.

Over the past decades, many researchers also have focused on improving the calculation performance of DDA for applications in geotechnical engineering (Chen et al., 1997; Cheng et al., 2000; Fu et al., 2016). With those improvements, it has been applied to solve various engineering problems such as rockfall (Chen, 2003, 2013, 2018; Ma et al., 2011; Wu et al., 2005), landslides (Sitar et al., 2005; Wu et al., 2007, 2009, 2010, 2011; Zhang et al., 2013, 2014, 2015; Chen et al., 2021), tunnels (Yeung and Leong, 1997; Wu et al., 2004; Tsesarsky and Hatzor, 2006; Do et al., 2020a, 2020b), seismic responses (Kamai and Hatzor, 2008; Zhang et al., 2014; Peng et al., 2018), and others (Song et al., 2017; Yu et al., 2019; Xia et

al., 2021). For example, Chen (2018) proposed an approach for rockburst analysis using an improved DDA program, which overcomes the challenge due to a dynamic and large deformation process in rockburst failure. Zhang (2015) extended the original DDA to simulate the Donghekou rockslide triggered by the 2008 Wenchuan earthquake, considering cohesion-frictional characteristics. To study the stability of tunnel roofs in a blocky rock mass, Tsesarsky and Hatzor (2006) carried out a kinematic analysis of the rock deformation as a function of joint spacing and friction using the DDA method. Peng et al. (2018) analyzed seismic wave propagation through discontinuous media by applying modified DDA to assess the dynamic response of sites containing discontinuities. Yu et al. (2019) estimated the impact force of rock landslides acting on structures for hazard prevention using the improved DDA method. DDA has various application in solving engineering problems and is an important tool for numerical simulation, such as the distinct element method and the finite element method. 3D DDA shares similar theory with the 2D version, however many problems in 3D DDA remains unsolved and it is difficult to analyze the complex earthquake-induced landslide problem using 3D DDA at present. In this study, the introduction in this section will focus on 2D DDA, because the mechanic analysis of earthquake-induced landslide is conducted by use of 2D DDA.

2.2. 2D DDA: STATE-OF-THE-ART

2.2.1. FORMULATIONS OF DDA

The DDA method is based on block theory, where individually deformable blocks can move independently, and the interaction between contacting blocks is realized by applying the contact springs. In 2D DDA, six main variables (u_0 , v_0 , r_0 , ε_x , ε_y , γ_{xy}), including rigid-body translations at the block center in two orthotropic directions, linear strains in two orthotropic directions, a rotation angle at block center and a shear strain, are defined for the rigid body displacements and uniform strains of each block. Therefore, the deformation matrix $\mathbf{D}_i$ of the block i can be expressed as

$$\mathbf{D}_i = (u_0 \quad v_0 \quad r_0 \quad \varepsilon_x \quad \varepsilon_y \quad \varepsilon_{xy})^T \quad (2.1)$$

where u_0 and v_0 are the rigid body translations of centroid point (x_0, y_0) within the block, and r_0 is the rotation angle of the block with the rotation center at (x_0, y_0) . ε_x , ε_y , γ_{xy} are the normal and shear strains of block i .

By using the displacement values obtained by the variables $(u_0, v_0, r_0, \varepsilon_x, \varepsilon_y, \gamma_{xy})$, the total displacement $\mathbf{U}$ of a point $P(x, y)$ within block i is obtained. Based on the matrix computation of deformation, the displacement $\mathbf{U}$ of a point P can be obtained as:

$$\mathbf{U} = \mathbf{T}_i \mathbf{D}_i \quad (2.2)$$

where $\mathbf{T}_i$ denotes displacement transformation matrix summing the translation, rotation and deformation of the block i . $\mathbf{U}$ and $\mathbf{T}_i$ can be respectively expressed as

$$\mathbf{U} = (u \quad v)^T \quad (2.3)$$

$$\mathbf{T}_i = \begin{pmatrix} 1 & 0 & -(y-y_0) & (x-x_0) & 0 & (y-y_0)/2 \\ 0 & 1 & (x-x_0) & 0 & (y-y_0) & (x-x_0)/2 \end{pmatrix} \quad (2.4)$$

The simultaneous equilibrium equations of block system are then derived by minimizing the total potential energy of the system, which can be expressed as:

$$\mathbf{M} \cdot \ddot{\mathbf{D}} + \mathbf{C} \cdot \dot{\mathbf{D}} + \mathbf{K} \cdot \mathbf{D} = \mathbf{F}(t, \mathbf{D}) \quad (2.5)$$

where $\mathbf{M}$, $\mathbf{C}$, and $\mathbf{K}$ are the mass matrix, damping matrix, and stiffness matrix of a block system with the external force vector $\mathbf{F}(t, \mathbf{D})$; $\ddot{\mathbf{D}}$, $\dot{\mathbf{D}}$, and $\mathbf{D}$ respectively denote the acceleration vector, velocity vector, and displacement vector. Denote that $\mathbf{D}_n$ and $\mathbf{D}_{n+1}$ respectively represent the approximation to $\mathbf{D}(t)$ and $\mathbf{D}(t+\Delta t)$ in a time step Δt . Then, Eq. (2.5) can be rewritten as:

$$\mathbf{M} \cdot \ddot{\mathbf{D}}_{n+1} + \mathbf{C} \cdot \dot{\mathbf{D}}_{n+1} + \mathbf{K} \cdot \mathbf{D}_{n+1} = \mathbf{F}_{n+1} \quad (2.6)$$

for initial conditions $\mathbf{D}(0)$ and $\dot{\mathbf{D}}(0)$, it is

$$\mathbf{D}(0) = 0, \quad (2.7)$$

$$\text{and } \dot{\mathbf{D}}(0) = 0. \quad (2.8)$$

In the original DDA (Shi, G. H. et al., 1989), time integration is performed by defining the potential energy associated with an increment of time. The scheme is equivalent to

$$\mathbf{D}_{n+1} = \mathbf{D}_n + \dot{\mathbf{D}}_n \Delta t + \frac{1}{2} \ddot{\mathbf{D}}_{n+1} \Delta t^2 + O(\Delta t^3) \quad (2.9)$$

Taking $\mathbf{D}_n = 0$ and solving for $\ddot{\mathbf{D}}_{n+1}$, it has

$$\frac{\Delta t^2}{2} \ddot{\mathbf{D}}_{n+1} \approx \mathbf{D}_{n+1} - \mathbf{D}_n - \dot{\mathbf{D}}_n \Delta t \quad (2.10)$$

$$\ddot{\mathbf{D}}_{n+1} \approx \frac{2}{\Delta t^2} \mathbf{D}_{n+1} - \frac{2}{\Delta t} \dot{\mathbf{D}}_n \quad (2.11)$$

As noted by Wang et al. (1996), this formulation is similar to the Newmark method, with parameters $\gamma = 1$ and $\beta = 1/2$ respectively for velocity weighting and acceleration:

$$\mathbf{D}_{n+1} = \mathbf{D}_n + \Delta t \dot{\mathbf{D}}_n + \frac{\Delta t^2}{2} \{ (1 - 2\beta) \ddot{\mathbf{D}}_n + 2\beta \ddot{\mathbf{D}}_{n+1} \} \quad (2.12)$$

$$\dot{\mathbf{D}}_{n+1} = \dot{\mathbf{D}}_n + \Delta t \{ (1 - \gamma) \ddot{\mathbf{D}}_n + \gamma \ddot{\mathbf{D}}_{n+1} \} \quad (2.13)$$

According to Eq. (2.12) and (2.13), the accelerations and velocities at the end of the time step can be obtained. Note that the displacements at the start of the current time step $\mathbf{D}_n = 0$ are applied to satisfy stepwise linearity and avoid algorithmic difficulties. The accelerations and velocities at the end of the time step can be respectively derived as

$$\ddot{\mathbf{D}}_{n+1} = \frac{2}{\Delta t^2} \mathbf{D}_{n+1} - \frac{2}{\Delta t} \dot{\mathbf{D}}_n \quad (2.14)$$

$$\dot{\mathbf{D}}_{n+1} = \dot{\mathbf{D}}_n + \Delta t \ddot{\mathbf{D}}_{n+1} = \frac{2}{\Delta t} \mathbf{D}_{n+1} - \dot{\mathbf{D}}_n \quad (2.15)$$

Substituting Eq. (2.14) and (2.15) into Eq. (2.6) gets the system of equations for solving the dynamic problem:

$$\left(\frac{2}{\Delta t^2} \mathbf{M} + \frac{2}{\Delta t} \mathbf{C} + \mathbf{K} \right) \cdot \mathbf{D}_{n+1} = \mathbf{F}_{n+1} + \left(\frac{2}{\Delta t} \mathbf{M} + \mathbf{C} \right) \cdot \dot{\mathbf{D}}_{n+1} \quad (2.16)$$

Collecting terms on both sides, the DDA system of equations may be conveniently expressed as

$$\hat{\mathbf{K}}\mathbf{D}_{n+1} = \hat{\mathbf{F}} \quad (2.17)$$

where $\hat{\mathbf{K}}$ and $\hat{\mathbf{F}}$ respectively denote the generalized stiffness matrix and the generalized load matrix, expressed as

$$\hat{\mathbf{K}} = \frac{2}{\Delta t^2} \mathbf{M} + \frac{2}{\Delta t} \mathbf{C} + \mathbf{K} \quad (2.18)$$

$$\hat{\mathbf{F}} = \mathbf{F}_{n+1} + \left(\frac{2}{\Delta t} \mathbf{M} + \mathbf{C} \right) \cdot \dot{\mathbf{D}}_{n+1} \quad (2.19)$$

The DDA system of equations can also be rewritten in matrix form as

$$\begin{bmatrix} \mathbf{K}_{11} & \mathbf{K}_{12} & \cdots & \mathbf{K}_{1n} \\ \mathbf{K}_{21} & \mathbf{K}_{22} & \cdots & \mathbf{K}_{2n} \\ \vdots & \vdots & \ddots & \vdots \\ \mathbf{K}_{n1} & \mathbf{K}_{n2} & \cdots & \mathbf{K}_{nn} \end{bmatrix} \begin{bmatrix} \mathbf{D}_1 \\ \mathbf{D}_2 \\ \vdots \\ \mathbf{D}_n \end{bmatrix} = \begin{bmatrix} \mathbf{F}_1 \\ \mathbf{F}_2 \\ \vdots \\ \mathbf{F}_n \end{bmatrix} \quad (2.20)$$

where $\mathbf{K}_{ij}$ ($i, j = 1, 2, \dots, n$) are a 6×6 coefficient submatrices, and $\mathbf{K}_{ii}$ are resulted from the block material properties while $\mathbf{K}_{ij}$ ($i \neq j$) depend on interactions between blocks i and j ; $\mathbf{D}_i$ ($i = 1, 2, \dots, n$) are the deformation matrix of block i ; $\mathbf{F}_i$ ($i = 1, 2, \dots, n$) are 6×1 loading submatrix on block i ; and n is the number of blocks in the defined block system.

The principle of minimum potential energy is adopted to calculate the matrices $\mathbf{K}_{ij}$ and $\mathbf{F}_i$ by taking the derivatives of the potential energy of every source with respect to each variable. In the DDA system, the total potential energy for a block system comes from the elastic stresses, initial constant stresses, body forces, point loadings, point displacement constraints, inertia forces, and contact forces. If the potential energy of a block system is Π , then

$$\Pi = \int_{\Omega} \vec{f} \cdot \vec{s} dx \quad (2.21)$$

where Ω denotes the domain of integration, $\vec{f}$ and $\vec{s}$ are the force vector that contributes to the potential energy and the corresponding displacement vector. Then, the contributions of energy source to the governing equations can be given by

$$\frac{\partial^2 \Pi}{\partial \mathbf{D}_i^2} \rightarrow \mathbf{K}_{ii} \quad (2.22)$$

$$\frac{\partial^2 \Pi}{\partial \mathbf{D}_j^2} \rightarrow \mathbf{K}_{jj} \quad (2.23)$$

$$\frac{\partial^2 \Pi}{\partial \mathbf{D}_i \partial \mathbf{D}_j} \rightarrow \mathbf{K}_{ij} \quad (2.24)$$

$$\frac{\partial^2 \Pi}{\partial \mathbf{D}_j \partial \mathbf{D}_i} \rightarrow \mathbf{K}_{ji} \quad (2.25)$$

$$-\frac{\partial \Pi(0)}{\partial \mathbf{D}_i} \rightarrow \mathbf{F}_i \quad (2.26)$$

$$-\frac{\partial \Pi(0)}{\partial \mathbf{D}_j} \rightarrow \mathbf{F}_j \quad (2.27)$$

where the subscripts i and j denote the indexes of both involved blocks; subscripts ii , jj , ij and ji indicate the position of the submatrices. It can be seen that the matrix $\mathbf{K}$ is symmetric because the matrix $\mathbf{K}_{ij} = \mathbf{K}_{ji}$.

2.2.2. NON-CONTACT MATRICES

In DDA method, the non-contact matrices include elastic stress, initial stress, body force, point loading, point displacement constrain, and inertia force. The method to obtain the matrices are similar: obtain the potential energy first, then take derivatives with respect to each variable. To unify the symbols, displacement vector in block i is rewritten as

$$\mathbf{D}_i = [d_{1i} \ d_{2i} \ d_{3i} \ d_{4i} \ d_{5i} \ d_{6i}]^T \quad (2.28)$$

2.2.2.1. ELASTIC STRESS

For each displacement step, assume that the blocks are linear elastic. Regarding block i , the elastic stress is a time-dependent variable. Since constant stress is assumed within a block, it is no longer a function to coordinate. The potential energy Π_e of elastic stress is computed by

$$\Pi_e = \iint \frac{1}{2} (\boldsymbol{\varepsilon}^T \cdot \boldsymbol{\sigma}) dx dy \quad (2.29)$$

where the integration is over the entire area of the i -th block, $\boldsymbol{\varepsilon}$ and $\boldsymbol{\sigma}$ are the strain and stress vector:

$$\boldsymbol{\varepsilon} = (\varepsilon_x \quad \varepsilon_y \quad \gamma_{xy})^T \quad (2.30)$$

$$\boldsymbol{\sigma} = (\sigma_x \quad \sigma_y \quad \tau_{xy})^T \quad (2.31)$$

The blocks in DDA are elastic, a physical equation is given below:

$$\boldsymbol{\sigma} = \mathbf{E} \cdot \boldsymbol{\varepsilon} \quad (2.32)$$

where $\mathbf{E}$ is expressed by

$$\mathbf{E} = \frac{E}{(1+\nu)(1-\nu)} \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & \nu & 0 \\ 0 & 0 & 0 & \nu & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1-\nu}{2} \end{bmatrix} \quad (2.33)$$

Then, stress vector can be expressed as

$$\boldsymbol{\sigma} = \frac{E}{(1+\nu)(1-\nu)} \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & \nu & 0 \\ 0 & 0 & 0 & \nu & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & (1-\nu)/2 \end{bmatrix} \begin{bmatrix} 0 \\ 0 \\ 0 \\ \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} \quad (2.34)$$

where E is Young's modulus, ν is Poisson's ratio.

The strain energy Π_e of elastic stress of block i in Eq. (2.29) can be expressed by using the block deformation:

$$\begin{aligned}
\Pi_e &= \iint \frac{1}{2} \begin{pmatrix} \varepsilon_x & \varepsilon_y & \gamma_{xy} \end{pmatrix} \begin{pmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{pmatrix} dx dy \\
&= \frac{1}{2} \iint [\mathbf{D}_i]^T [\mathbf{E}_i] [\mathbf{D}_i] dx dy \\
&= \frac{S_{i,0}}{2} [\mathbf{D}_i]^T [\mathbf{E}_i] [\mathbf{D}_i]
\end{aligned} \tag{2.35}$$

where $S_{i,0}$ is the entire area of block i .

The derivatives are computed to minimize the strain energy Π_e :

$$k_{rs} = \frac{\partial^2 \Pi_e}{\partial d_{ri} \partial d_{rs}} = \frac{S_{i,0}}{2} \frac{\partial^2}{\partial d_{ri} \partial d_{rs}} \left([\mathbf{D}_i]^T [\mathbf{E}_i] [\mathbf{D}_i] \right), \quad r, s = 1, \dots, 6 \tag{2.36}$$

k_{rs} forms a 6×6 submatrix:

$$S_{i,0} \mathbf{E}_i \rightarrow \mathbf{K}_{ii} \tag{2.37}$$

The submatrix of Eq. (2.37) will be added to the matrix $\mathbf{K}_{ii}$ in the global stiffness equation.

2.2.2.2. INITIAL STRESS

Initial stress is constant. For block i , the initial stress $\boldsymbol{\sigma}_0$ is predefined as

$$\boldsymbol{\sigma}_0 = \begin{bmatrix} 0 & 0 & 0 & \sigma_x^0 & \sigma_y^0 & \tau_{xy}^0 \end{bmatrix}^T \tag{2.38}$$

The potential energy of the initial stresses Π_σ is expressed by

$$\Pi_\sigma = \iint (\boldsymbol{\varepsilon}^T \cdot \boldsymbol{\sigma}_0) dx dy = S_{i,0} [\mathbf{D}_i]^T [\boldsymbol{\sigma}_0] \tag{2.39}$$

The derivatives of Π_σ are computed to minimize potential energy of the initial stresses. Because $[\mathbf{D}_i]^T$ is a one-order term, the body force does not contribute to the global stiffness matrix, and only force submatrix is obtained:

$$f_r = -\frac{\partial \Pi_\sigma(0)}{\partial d_{ri}} = -S_{i,0} \frac{\partial [\mathbf{D}_i]^T [\boldsymbol{\sigma}_0]}{\partial d_{ri}}, \quad r = 1, \dots, 6 \tag{2.40}$$

f_r forms a 6×1 submatrix:

$$-S_{i,0} \boldsymbol{\sigma}_0 \rightarrow \mathbf{F}_i \tag{2.41}$$

The submatrix of Eq. (2.41) will be added to $\mathbf{F}_i$ in the global equations.

2.2.2.3. BODY FORCE

The body force denotes the volume loading (f_x, f_y) which consists of functions of coordinates (x, y) acting on the block i . The potential energy of the body loading Π_v can be expressed as

$$\begin{aligned}\Pi_v &= - \iint (u \quad v) \begin{pmatrix} f_x \\ f_y \end{pmatrix} dx dy \\ &= -S_{i,0} [\mathbf{D}_i]^T \begin{bmatrix} f_x & f_y & 0 & 0 & 0 & 0 \end{bmatrix}^T\end{aligned}\quad (2.42)$$

where f_x and f_y respectively represent the components x -, and y - directions of body force. Since no second-order term of $[\mathbf{D}_i]^T$ exists, the body force does not contribute to the global stiffness matrix. Only force submatrix is obtained:

$$f_r = -\frac{\partial \Pi_v(0)}{\partial d_{ri}} = \frac{\partial}{\partial d_{ri}} \left([\mathbf{D}_i]^T \begin{bmatrix} f_x S_{i,0} & f_y S_{i,0} & 0 & 0 & 0 & 0 \end{bmatrix}^T \right), \quad r=1, \dots, 6 \quad (2.43)$$

f_r forms a 6×1 submatrix:

$$\begin{bmatrix} f_x S_{i,0} & f_y S_{i,0} & 0 & 0 & 0 & 0 \end{bmatrix}^T \rightarrow \mathbf{F}_i \quad (2.44)$$

The submatrix of Eq. (2.44) will be added to $\mathbf{F}_i$ in the global equations.

2.2.2.4. POINT LOADING

If a point loading force $\vec{f}_n = [F_x \quad F_y]^T$ is exerted on point $P(x, y)$ within block i , the potential energy of the point loading Π_p can be expressed as

$$\Pi_p = -[u \quad v] \cdot [F_x \quad F_y]^T = -[\mathbf{D}_i]^T [\mathbf{T}_i]^T [F_x \quad F_y]^T \quad (2.45)$$

The derivatives of Π_p are computed to minimize potential energy of the initial stresses:

$$f_r = -\frac{\partial \Pi_p(0)}{\partial d_{ri}} = \frac{\partial}{\partial d_{ri}} [\mathbf{D}_i]^T [\mathbf{T}_i]^T [F_x \quad F_y]^T, \quad r=1, \dots, 6 \quad (2.46)$$

f_r forms a 6×1 submatrix:

$$\mathbf{T}_i^T \begin{bmatrix} F_x & F_y \end{bmatrix}^T \rightarrow \mathbf{F}_i \quad (2.47)$$

The submatrix of Eq. (2.47) will be added to $\mathbf{F}_i$ in the global equations.

2.2.2.5. POINT DISPLACEMENT CONSTRAIN

In DDA method, the point displacement constrain is set by implementing a very stiff constrain spring to change the displacement (u, v) of point $P(x, y)$ within block i , to (u_m, v_m) . When $u_m = v_m = 0$, the constrain is performs as a fixed point. The potential energy Π_m is expressed as

$$\begin{aligned}\Pi_m &= \frac{p}{2}[(u - u_m)^2 + (v - v_m)^2] \\ &= \frac{p}{2}[\mathbf{D}_i]^T [\mathbf{T}_i]^T [\mathbf{T}_i] [\mathbf{D}_i] - p[\mathbf{D}_i]^T [\mathbf{T}_i]^T \begin{bmatrix} u_m \\ v_m \end{bmatrix} + p[u_m \quad v_m] \cdot \begin{bmatrix} u_m \\ v_m \end{bmatrix}\end{aligned}\quad (2.48)$$

where p is the stiffness of the constrain spring.

The derivatives of Π_m are computed to minimize potential energy of the initial stresses, the contributions to the generalized stiffness submatrix and the generalized force submatrix can be derived as

$$p[\mathbf{T}_i]^T [\mathbf{T}_i] \rightarrow \mathbf{K}_{ii} \quad (2.49)$$

$$p[\mathbf{T}_i]^T [u_m \quad v_m]^T \rightarrow \mathbf{F}_i \quad (2.50)$$

The submatrices of Eq. (2.49) and Eq. (2.50) will be respectively added to $\mathbf{K}_{ii}$ and $\mathbf{F}_i$ in the global equations.

2.2.2.6. INERTIAL FORCE

In 2D DDA method, assuming that $(u(t), v(t))$ are the time-dependent displacement of any point (x, y) of block i , the force of inertia can be expressed as

$$\vec{f}_n = \begin{bmatrix} f_x & f_y \end{bmatrix}^T = -\rho \frac{\partial^2 \mathbf{U}}{\partial t^2} = -\rho [\mathbf{T}_i] \cdot \frac{\partial^2 [\mathbf{D}_i]}{\partial t^2} \quad (2.51)$$

where f_x and f_y are the components of inertial force in x -, and y - directions; ρ is the mass per unit volume; t is time.

The potential energy Π_i of the inertia force of block i is

$$\begin{aligned}
\Pi_i &= - \iint U \cdot \vec{f}_n dx dy \\
&= \iint [\mathbf{D}_i^T \cdot \mathbf{T}_i^T] \cdot [\rho \mathbf{T}_i \cdot \frac{\partial^2 \mathbf{D}_i}{\partial t^2}] dx dy \\
&= \rho \mathbf{D}_i^T \cdot \iint \mathbf{T}_i^T \cdot \mathbf{T}_i dx dy \cdot \frac{\partial^2 \mathbf{D}_i}{\partial t^2}
\end{aligned} \tag{2.52}$$

Denote $\mathbf{D}_i(t) = 0$ as the displacement at the beginning of the current time step, and the displacement at the end of the current time step $\mathbf{D}_i(t + \Delta t) = \mathbf{D}_i$. Δt is the time step. By the time integration, it results in

$$\begin{aligned}
\mathbf{D}_i &= \mathbf{D}_i(t + \Delta t) \\
&= \mathbf{D}_i(t) + \Delta t \frac{\partial \mathbf{D}_i(t)}{\partial t} + \frac{(\Delta t)^2}{2} \frac{\partial^2 \mathbf{D}_i(t)}{\partial t^2} \\
&= \Delta t \frac{\partial \mathbf{D}_i(t)}{\partial t} + \frac{(\Delta t)^2}{2} \frac{\partial^2 \mathbf{D}_i(t)}{\partial t^2}
\end{aligned} \tag{2.53}$$

Assuming that the acceleration is constant at a single time step Δt , according to Eq. (2.53), it has

$$\frac{\partial^2 \mathbf{D}_i(t + \Delta t)}{\partial t^2} = \frac{\partial^2 \mathbf{D}_i(t)}{\partial t^2} = \frac{2}{(\Delta t)^2} \mathbf{D}_i - \frac{2}{\Delta t} \mathbf{V}_i(t) \tag{2.54}$$

where $\mathbf{V}_i(t) = \partial \mathbf{D}_i(t) / \partial t$.

The potential energy Π_i of the inertia force of block i at the end of the current time step can be repressed as

$$\begin{aligned}
\Pi_i &= \rho \mathbf{D}_i^T \iint \mathbf{T}_i^T \cdot \mathbf{T}_i dx dy \cdot \frac{\partial^2 \mathbf{D}_i}{\partial t^2} \\
&= \rho \mathbf{D}_i^T \iint \mathbf{T}_i^T \cdot \mathbf{T}_i dx dy \cdot \left(\frac{2}{(\Delta t)^2} \mathbf{D}_i - \frac{2}{\Delta t} \mathbf{V}_i(t) \right)
\end{aligned} \tag{2.55}$$

As only $\mathbf{D}_i$ is unknown in Eq. (2.55), taking derivatives for $\mathbf{D}_i$ to minimize potential energy of the inertia force, the contributions to the generalized stiffness submatrix and the generalized force submatrix can be derived as

$$\frac{2\rho}{\Delta t^2} \iint \mathbf{T}_i^T \cdot \mathbf{T}_i dx dy \rightarrow \mathbf{K}_{ii} \tag{2.56}$$

$$\frac{2\rho}{\Delta t} \iint \mathbf{T}_i^T \cdot \mathbf{T}_i dx dy \cdot \mathbf{V}_i \rightarrow \mathbf{F}_i \tag{2.57}$$

The submatrices of Eq. (2.56) and Eq. (2.57) will be respectively added to $\mathbf{K}_{ii}$ and $\mathbf{F}_i$ in the global equations. The integrations in the above two equations can be addressed as follows:

$$\iint \mathbf{T}_i^T \cdot \mathbf{T}_i dx dy = \begin{bmatrix} S & 0 & 0 & 0 & 0 & 0 \\ 0 & S & 0 & 0 & 0 & 0 \\ 0 & 0 & S_1 + S_2 & -S_3 & S_3 & \frac{S_1 - S_2}{2} \\ 0 & 0 & -S_3 & S_1 & 0 & \frac{S_3}{2} \\ 0 & 0 & S_3 & 0 & S_2 & \frac{S_3}{2} \\ 0 & 0 & \frac{S_1 - S_2}{2} & \frac{S_3}{2} & \frac{S_3}{2} & \frac{S_1 + S_2}{2} \end{bmatrix} \quad (2.58)$$

where

$$\begin{cases} S_1 = S_{xx} - x_0 S_x \\ S_2 = S_{yy} - y_0 S_y \\ S_3 = S_{xy} - x_0 S_y \end{cases} \quad (2.59)$$

in which

$$\begin{cases} x_0 = S_x / S \\ y_0 = S_y / S \end{cases} \quad (2.60)$$

and

$$\begin{cases} S = \iint dx dy \\ S_x = \iint x dx dy \\ S_y = \iint y dx dy \\ S_{xx} = \iint x^2 dx dy \\ S_{yy} = \iint y^2 dx dy \\ S_{xy} = \iint xy dx dy \end{cases} \quad (2.61)$$

where the values of S , S_x , S_y , S_{xx} , S_{yy} , S_{xy} can be calculated by using the 2D simplex integration.

2.2.3. CONTACT MECHANISM AND CONTACT MATRICES

2.2.3.1. CONTACT TYPES AND DETECTION

In 2D DDA method, interaction forces between blocks are calculated based on contact mechanism. In general, the contact between polygonal blocks can be handled by contact detection and contact treatment. Contact detection in DDA includes two steps, namely rough contact search and precise contact detection. At first step, DDA will find all possible contacts in the current step, referred to as rough contact search. Several strategies are usually employed in DDA to conduct rough contact search, including no binary search (NBS), double-ended spatial sorting (DESS), multi-shell cover (MSC) and multi-cover searching (MCS). In the second step, the identified possible contact from the first step will be examined in detail to define the contact types and then determine the entrance mode for each contact. This is referred to as precise contact detection, which determines the exact contact positions and types between blocks. In the 2D DDA method, three types of contacts are defined for contact detection, including vertex-to-vertex (V-V), vertex-to-edge (V-E), and edge-to-edge (E-E), as shown in Figure 2.1. As an edge-to-edge contact can be treated as two vertex-to-edge contacts, only two contact types, vertex-to-edge and vertex-to-vertex, are calculated in the DDA. The direct contact detection algorithm based on the distance criterion and angle method will be used for precise contact detection in this step.

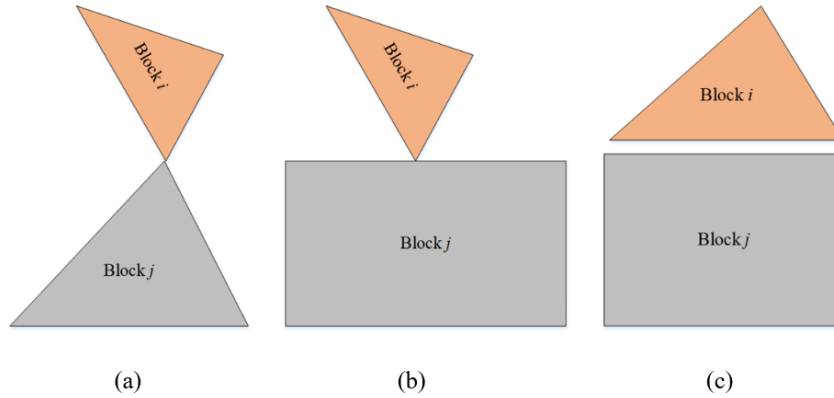


Figure 2.1 Three types of contact in DDA: (a) vertex-to-vertex (b) vertex-to-edge (c) edge-to-edge

For a block system, after all the contact positions and types are determined, proper contact springs are added to each contact to prevent the blocks from penetrating each other, called contact treatment. In the penalty method, normal and shear contact springs are employed based on the actual contact situation to act as contact constraints between blocks. The open-close iteration process (OCI) is implemented in each time step to achieve the contact state convergence between blocks. Three types of contact states (locked, sliding, or open) between blocks can be determined during OCI, in which the separation is defined as the 'open' state and the contact with 'locked' state or 'sliding' state is defined as the 'close' state. For the locked contact state, both normal and shear springs are applied; for the sliding contact state, normal spring and frictional force are applied; for the open contact state, no interaction force is applied. The contact state of one contact is deduced before calculation and revised according to the calculated results. Open-close iteration and contact submatrices in 2D DDA will be introduced in the following sections.

2.2.3.2. OPEN-CLOSE ITERATION (OCI)

The process in contact treatment to realize the satisfaction of the contact conditions at the end of a time step is called open-close iteration (OCI), which includes solving simultaneous equations and the successive checking of interpenetration. After solving the governing equations, an OCI is then

implemented to determine the valid system state. The final states of all contacts after solving the governing equations are tested and compared to those assumed before. The system state is considered valid only if all the calculated contact states are consistent with the initial ones. The OCI follows the criteria that the shear spring force cannot exceed the maximum friction, and the distance serves as evidence to determine open and close:

$$k_n d_n > 0 \quad \text{and} \quad k_\tau d_\tau \leq k_n d_n \tan \varphi + \tau_c l \rightarrow \text{locked} \quad (2.62)$$

$$k_n d_n > 0 \quad \text{and} \quad k_\tau d_\tau > k_n d_n \tan \varphi + \tau_c l \rightarrow \text{sliding} \quad (2.63)$$

$$k_n d_n < -\sigma_c l \rightarrow \text{open} \quad (2.64)$$

where τ_c , φ , σ_c and l are cohesion, friction angle, tensile stress and contact length of the contact. k_n and k_τ respectively denotes the stiffness of normal spring and shear spring. d_n is the normal penetration distance and d_τ is the tangential relative displacement.

2.2.3.3. NORMAL SPRING SUBMATRIX

Assume there is a point P_1 on block i penetrating block j by reference line P_2P_3 . Denote (x_i, y_i) and (u_i, v_i) as the coordinates and displacement increment of point P_i respectively, $i = 1, 2, 3$. A spring between point P_1 and reference line P_2P_3 will be applied to push point P_1 back from inter-penetration line P_2P_3 in normal direction. The distance from point P_1 to line P_2P_3 is defined as the entrance distance d_n' , which can be calculated by

$$d_n' \approx \frac{V}{A_0} + \frac{1}{A_0} \left(\begin{vmatrix} 1 & u_1 & y_1 \\ 1 & u_2 & y_2 \\ 1 & u_3 & y_3 \end{vmatrix} + \begin{vmatrix} 1 & x_1 & v_1 \\ 1 & x_2 & v_2 \\ 1 & x_3 & v_3 \end{vmatrix} + \begin{vmatrix} 1 & x_1 & y_1 \\ 1 & x_2 & y_2 \\ 1 & x_3 & y_3 \end{vmatrix} \right) \quad (2.65)$$

where

$$A_0 = |\overline{P_2P_3}| = \sqrt{(x_3 - x_2)^2 + (y_3 - y_2)^2} \quad (2.66)$$

$$V = \begin{vmatrix} v_{11} & v_{12} & v_{13} \\ v_{21} & v_{22} & v_{23} \\ v_{31} & v_{32} & v_{33} \end{vmatrix} = \begin{vmatrix} 1 & x_1 & y_1 \\ 1 & x_2 & y_2 \\ 1 & x_3 & y_3 \end{vmatrix} \quad (2.67)$$

Denote L_{rs} as the algebraic complement of determinant V about element v_{rs} , L_{rs} can be rewritten as:

$$L_{rs} = (-1)^{r+s} V_{rs} \quad (2.68)$$

Then, the Eq. (2.65) can be expressed as

$$d_n' \approx d_n + \frac{1}{A_0} [L_{12} \quad L_{13}] \begin{bmatrix} u_1 \\ v_1 \end{bmatrix} + \frac{1}{A_0} \sum_{r=2}^3 \left([L_{r2} \quad L_{r3}] \begin{bmatrix} u_r \\ v_r \end{bmatrix} \right) \quad (2.69)$$

where

$$d_n = \left(\sum_{r=1}^3 L_{r1} \right) / A_0 \quad (2.70)$$

Substituting $\mathbf{D}_i$ into Eq. (2.69) leads to

$$d_n' \approx d_n + \mathbf{H} \cdot \mathbf{D}_i + \mathbf{G} \cdot \mathbf{D}_j \quad (2.71)$$

where $\mathbf{H}$ and $\mathbf{G}$ denote coefficient matrices, respectively derived as

$$\mathbf{H} = \frac{1}{A_0} [L_{12} \quad L_{13}] \cdot \mathbf{T}_i(x_1, y_1) \quad (2.72)$$

$$\mathbf{G} = \frac{1}{A_0} \sum_{r=2}^3 [L_{r2} \quad L_{r3}] \cdot \mathbf{T}_j(x_r, y_r) \quad (2.73)$$

With the stiffness of the spring k_n and the inter-penetration distance d_n' , the potential strain energy Π_n of the contact spring can be calculated by

$$\begin{aligned} \Pi_n &= \frac{k_n}{2} (d_n')^2 = \frac{k_n}{2} (d_n + \mathbf{H} \cdot \mathbf{D}_i + \mathbf{G} \cdot \mathbf{D}_j)^2 \\ &= \frac{k_n}{2} (d_n^2 + 2d_n \mathbf{H} \cdot \mathbf{D}_i + 2d_n \mathbf{G} \cdot \mathbf{D}_j + \mathbf{H} \cdot \mathbf{D}_i \cdot \mathbf{D}_i^T \cdot \mathbf{H}^T \\ &\quad + \mathbf{G} \cdot \mathbf{D}_j \cdot \mathbf{D}_j^T \cdot \mathbf{G}^T + 2\mathbf{H} \cdot \mathbf{D}_i \cdot \mathbf{D}_j^T \cdot \mathbf{G}^T) \end{aligned} \quad (2.74)$$

Taking derivatives of potential strain energy Π_n of the normal spring yields the generalized stiffness matrices and the generalized load matrices added to the global stiffness and force matrices:

$$k_n \mathbf{H}^T \cdot \mathbf{H} \rightarrow \mathbf{K}_{ii} \quad (2.75)$$

$$k_n \mathbf{G}^T \cdot \mathbf{G} \rightarrow \mathbf{K}_{jj} \quad (2.76)$$

$$k_n \mathbf{H}^T \cdot \mathbf{G} \rightarrow \mathbf{K}_{ij} \quad (2.77)$$

$$k_n \mathbf{G}^T \cdot \mathbf{H} \rightarrow \mathbf{K}_{ji} \quad (2.78)$$

$$-k_n d_n \mathbf{H}^T \rightarrow \mathbf{F}_i \quad (2.79)$$

$$-k_n d_n \mathbf{G}^T \rightarrow \mathbf{F}_j \quad (2.80)$$

2.2.3.4. SHEAR SPRING SUBMATRIX

In 2D DDA method, the shear contact displacement d_τ' between a contact point pair can be calculated as

$$\begin{aligned} d_s' &\approx \frac{1}{\left| \overrightarrow{P_2 P_3} \right|} \overrightarrow{P_b^u P_a^u} \cdot \overrightarrow{P_2 P_3} \\ &= \frac{[x_a + u_a - (x_b + u_b) \quad y_a + v_a - (y_b + v_b)] \cdot [x_3 - x_2 \quad y_3 - y_2]^T}{\sqrt{(x_3 - x_2)^2 + (y_3 - y_2)^2}} \quad (2.81) \\ &= S_0 + [u_a - u_b \quad v_a - v_b] \cdot \mathbf{e}_{23} \end{aligned}$$

with

$$\mathbf{e}_{23} = \frac{1}{\sqrt{(x_3 - x_2)^2 + (y_3 - y_2)^2}} [x_3 - x_2 \quad y_3 - y_2]^T \quad (2.82)$$

$$S_0 = [x_a - x_b \quad y_a - y_b] \cdot \mathbf{e}_{23} \quad (2.83)$$

Similarly, the potential energy Π_s contributed by the shear spring with stiffness k_s can be calculated by

$$\begin{aligned} \Pi_s &= \frac{k_s}{2} (d_s^u)^2 = \frac{k_s}{2} (S_0 + [u_a - u_b \quad v_a - v_b] \cdot \mathbf{e}_{23})^2 \\ &= \frac{k_s S_0^2}{2} + k_s S_0 \mathbf{D}_i^T \cdot \mathbf{T}_i^T(x_a, y_a) \cdot \mathbf{e}_{23} - k_s S_0 \mathbf{D}_j^T \cdot \mathbf{T}_j^T(x_b, y_b) \cdot \mathbf{e}_{23} \\ &\quad + \frac{k_s}{2} \mathbf{D}_i^T \cdot \mathbf{T}_i^T(x_a, y_a) \cdot \mathbf{e}_{23} \cdot \mathbf{e}_{23}^T \cdot \mathbf{T}_i(x_a, y_a) \cdot \mathbf{D}_i \\ &\quad - \frac{k_s}{2} \mathbf{D}_i^T \cdot \mathbf{T}_i^T(x_a, y_a) \cdot \mathbf{e}_{23} \cdot \mathbf{e}_{23}^T \cdot \mathbf{T}_j(x_b, y_b) \cdot \mathbf{D}_j \\ &\quad - \frac{k_s}{2} \mathbf{D}_j^T \cdot \mathbf{T}_j^T(x_b, y_b) \cdot \mathbf{e}_{23} \cdot \mathbf{e}_{23}^T \cdot \mathbf{T}_i(x_a, y_a) \cdot \mathbf{D}_i \\ &\quad + \frac{k_s}{2} \mathbf{D}_j^T \cdot \mathbf{T}_j^T(x_b, y_b) \cdot \mathbf{e}_{23} \cdot \mathbf{e}_{23}^T \cdot \mathbf{T}_j(x_b, y_b) \cdot \mathbf{D}_j \end{aligned} \quad (2.84)$$

Taking derivatives of potential strain energy Π_s of the shear spring yields the generalized stiffness matrices and the generalized load matrices added to the global stiffness and force matrices:

$$k_s \mathbf{T}_i^T(x_a, y_a) \cdot \mathbf{e}_{23} \cdot \mathbf{e}_{23}^T \cdot \mathbf{T}_i(x_a, y_a)_i \rightarrow \mathbf{K}_{ii} \quad (2.85)$$

$$k_s \mathbf{T}_j^T(x_b, y_b) \cdot \mathbf{e}_{23} \cdot \mathbf{e}_{23}^T \cdot \mathbf{T}_j(x_b, y_b) \rightarrow \mathbf{K}_{jj} \quad (2.86)$$

$$-\frac{k_s}{2} \mathbf{T}_i^T(x_a, y_a) \cdot \mathbf{e}_{23} \cdot \mathbf{e}_{23}^T \cdot \mathbf{T}_j(x_b, y_b) \rightarrow \mathbf{K}_{ij} \quad (2.87)$$

$$-\frac{k_s}{2} \mathbf{T}_j^T(x_b, y_b) \cdot \mathbf{e}_{23} \cdot \mathbf{e}_{23}^T \cdot \mathbf{T}_i(x_a, y_a) \rightarrow \mathbf{K}_{ji} \quad (2.88)$$

$$-k_s S_0 \mathbf{T}_i^T(x_a, y_a) \cdot \mathbf{e}_{23} \rightarrow \mathbf{F}_i \quad (2.89)$$

$$k_s S_0 \mathbf{T}_j^T(x_b, y_b) \cdot \mathbf{e}_{23} \rightarrow \mathbf{F}_j \quad (2.90)$$

2.2.3.5. FRICTION SUBMATRIX

When contact state is ‘sliding’, the shear resistance exceeds the shear force from the shear spring. Then, shear contact spring will be removed and replaced by friction. In 2D DDA method, the Coulomb friction law is adopted to calculate friction. The shear force $\mathbf{F}_f$ imposed on the contact point P_a can be calculated as

$$\mathbf{F}_f = -|\mathbf{F}_n| \tan \phi \mathbf{s} = -k_n |d_n'| \tan \phi \boldsymbol{\tau} \quad (2.91)$$

where $\mathbf{F}_n$ is the normal force from the normal spring; ϕ denotes the friction angle; $\boldsymbol{\tau}$ indicates the unit vector of relative shear displacement of P_a with respect to P_b calculated in the previous iteration step or time step. The shear displacement in this step is expressed as

$$\mathbf{d}_f' = [u_a - u_b \quad v_a - v_b] \cdot \boldsymbol{\tau} \cdot \boldsymbol{\tau} \quad (2.92)$$

The potential energy Π_f of the frictional forces can be calculated by

$$\begin{aligned} \Pi_f &= \mathbf{F}_f \mathbf{d}_f' = |\mathbf{F}_f| [u_a - u_b \quad v_a - v_b] \cdot \boldsymbol{\tau} \\ &= |\mathbf{F}_f| [u_a \quad v_a] \cdot \boldsymbol{\tau} - |\mathbf{F}_f| [u_b \quad v_b] \cdot \boldsymbol{\tau} \end{aligned} \quad (2.93)$$

Submitting $\mathbf{D}_i$ into Eq. (2.93), the potential energy Π_f can be expressed as

$$\Pi_f = \left| \mathbf{F}_f \right| \mathbf{D}_i^T \cdot \mathbf{T}_i^T(x_a, y_a) \cdot \boldsymbol{\tau} - \left| \mathbf{F}_f \right| \mathbf{D}_j^T \cdot \mathbf{T}_j^T(x_b, y_b) \cdot \boldsymbol{\tau} \quad (2.94)$$

Taking derivatives of potential strain energy Π_f of the shear spring yields the generalized stiffness matrices and the generalized load matrices added to the global stiffness and force matrices:

$$-\left| \mathbf{F}_f \right| \mathbf{T}_i^T(x_a, y_a) \cdot \boldsymbol{\tau} \rightarrow \mathbf{F}_i \quad (2.95)$$

$$\left| \mathbf{F}_f \right| \mathbf{T}_j^T(x_b, y_b) \cdot \boldsymbol{\tau} \rightarrow \mathbf{F}_j \quad (2.96)$$

2.3. LANDSLIDE SIMULATION WITH DDA: STATE-OF-THE-ART

Earthquake is one of the most common factors triggering landslides. Over the past decades, a great number of catastrophic landslides were triggered during the historical earthquake events, such as 1995 Kobe earthquake, 1999 Chi-Chi earthquake, 2008 Wenchuan earthquake and 2016 Kumamoto earthquake. The investigation on the mechanism of landslide initiation and evaluation is important for earthquake disaster prevention and mitigation. Many numerical methods, including finite-difference method (FDM) (Mitchell and Griffiths, 1980), boundary element method (BEM) (Brebbia and Wrobel, 1980), discrete element method (DEM) (Cundall, 1971) and discontinuous deformation analysis (DDA) (Shi 1989), are proposed to simulate the landslide initiation and evaluation processes, which make great contributions to the study of landslide mechanism. DDA is a numerical method that has great advantage in simulating the large displacement and rigid body movements of the fractured and jointed rock masses. Since the DDA method was proposed, it has been widely applied for analyzing the landslide simulation. Over the past decades, lots of research on DDA have been conducted to obtain a more precise simulation of earthquake-induced landslides, such as solutions to the free expansion phenomenon (Wu et al., 2005, 2015; Jiang and Zheng, 2015; Gong et al., 2020), improvements on contact problems (Bao and Zhao, 2010, 2012; Fan and He, 2015; Ni et al., 2020), parallel computing (Cheng et al., 2015; Xiao et al., 2017; Song et al., 2017; Jiao et al., 2019; Peng et al., 2019a). Meanwhile, many extensions on DDA have been proposed to make it more available for simulating actual landslides consisting of various factors.

In recent years, 2D DDA has been greatly developed and applied in landslide simulation. Tsaoiling landslide is a large-scale earthquake-induced landslide in the 1999 Chi-Chi earthquake. [Wu and Chen \(2011\)](#) successfully simulated the post-failure behavior of the Tsaoiling landslide using the DDA method. The run-out distance of the landslide and final deposition topography determined by the DDA simulation coincide well with the actual post-failure landform. With application of DDA, [Zhang et al., \(2015\)](#) analyzed the Donghekou landslide considering the effect of seismic loading on the run-out of landslides. The results verified that the seismic loading could be one of the factors that could eventually help increase the run-out of landslides. [Song et al., \(2016\)](#) considered the friction degradation of the sliding surface during landslide movement and incorporated a velocity and displacement dependent friction law into DDA, which assist in further understanding the movement behaviors of high-speed and long run-out Daguangbao landslide induced by the 2008 Wenchuan earthquake. [Chen and Wu \(2018\)](#) simulated the failure process of the Xinmo landslide using DDA. This study demonstrates that the DDA accurately simulates the post-failure behavior and determines the impact area of a landslide with complicated failure processes and topography. [Ibanez et al., \(2018\)](#) implemented a friction degradation law into DDA to study velocity-distance evolution of the catastrophic Vajont landslide. Therefore, catastrophic landslides can become natural experiments at the field scale that provide ultimate friction degradation values for clay filled rock discontinuities. [Nian et al., \(2020\)](#) proposed a unified state-dependent shear strength model in DDA to describe the shear strength degradation in a seismic landslide process. Numerical simulations of the Daguangbao landslide triggered by the Wenchuan earthquake were performed and the results illustrate that the shear strength degradation of discontinuities affect the evolution process, travel distance, and post-failure shape of the seismic landslide significantly. [Peng et al., \(2020\)](#) implemented SPH into DDA code for simulating submarine landslides consisting of multiple blocks. The performance of the coupled DDA–SPH method is validated through a submarine rigid landslide, and the simulation results are in good agreement with the experimental data. In conclusion, DDA method has been successfully applied to analyze actual landslide occurred in

recent disasters and shows great applicability and feasibility in landslide simulation that considers various factors. By using the DDA method, many researches on the practical problems of landslide initiation and evaluation have been successfully conducted.

However, slope stability analysis under seismic conditions is still a big challenge. How to accurately consider the effect of seismic waves in slope stability analysis has not been well clarified, although many methods have been proposed for slope stability analysis under earthquake. One of the main difficulties is to determine the factor effectively representing earthquake intensity. The other is how to accurately consider the factor of the earthquake in the methods for slope stability analysis. DDA has a big advantage for analyzing slope stability analysis, and it has been widely applied to solving earthquake-induced landslides since it was proposed. In this study, to effectively analyze slope stability analysis under seismic condition, the contact processing and fracture models of original DDA is also improved and extended necessarily so as to be better applied for simulating the earthquake-induced landslides.

2.4. CONCLUSIONS

In this chapter, a brief introduction has been presented for both 2D DDA formulation and the development and application of 2D DDA in landslide simulation.

2D DDA, a discontinuous method, shows a great advantage in analyzing cases involving large deformation and ridge body movement of the discontinuous block system. At present, the calculation performance of 2D DDA has been significantly developed and improved, and it can be successfully applied to analyze actual landslides in various cases. However, there are still some problems unsolved for earthquake-induced landslide simulation in the pacificated cases, which will be illustrated in the following sections.

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CHAPTER 3

IMPROVEMENTS ON TWO-DIMENSIONAL DDA ¹

3.1. INTRODUCTION

Contact processing is a well-known complex problem in the DDA algorithm, and many studies that focused on contact problems have been conducted for 2D and 3D DDA since the DDA method has been proposed (Yeung et al., 2007; Keneti et al., 2008; Bao and Zhao, 2010, 2012; Ni et al., 2020a, 2020b; Zheng et al., 2020; Wang et al., 2019, 2020; Wu et al., 2014, 2020). In the original two-dimensional DDA, contact detection based on the parameters of distance and angle sometimes leads to an indeterminate state, as contact processing is unique and complex. In particular, for a block system that has a large number of strangely shaped blocks, the calculation result can be unreasonable. Since DDA is proposed by Dr. Shi (Shi, 1989), many researchers have worked on the original two-dimensional DDA contact problem to solve potential contact indeterminacy. Bao and Zhao (2010) firstly enhanced the algorithm for choosing the initial contact edge of the vertex-to-vertex contact by using a special contact spring. Then, they utilized the trajectory

¹ Chapter 3 is based on the following journal papers:

Xia M, Chen G, Yu P, Peng X, & Zou J (2021) Improvement of DDA with a New Unified Tensile Fracture Model for Rock Fragmentation and its Application on Dynamic Seismic Landslides. *Rock Mechanics and Rock Engineering*, 54(3), 1055-1075.

Xia M, Chen G, & Guo L (2021) Improvement of vertex-to-vertex contact processing in two-dimensional DDA. *Computers and Geotechnics*, 135, doi: 10.1016/j.compgeo.2021.104200.

of the vertex during a time step as the second enhancement for detecting the entrance edge (Bao and Zhao, 2012). However, if there is a gap between two vertices, Bao and Zhao's method can affect the motion trend of blocks because of the additional temporary contact springs. To address the genuine indeterminacy of vertex-to-vertex contact, Fan and He (2015) developed a new pre-processing and angle-based method to determine the entrance edges. Ni et al. (2020) proposed a contact detection algorithm for arbitrary polygonal blocks based on contact theory to improve contact detection accuracy. Four different methods were applied to determine the contact entrance lines based on the contact theory. Though it has proven to be an effective and strict method, it is highly time-consuming. Their research can partially solve the indeterminacy of contact, but it has limitations. It was found that the angle criterion still has limitations for contact processing, and there was a lost case of contact when detected using the angle criteria, which was then discovered in our study. Thus, a stricter algorithm for contact detection is necessary. Contact processing plays a vital role in accurately predicting the behavior of block interactions, which closely affects the safety and stability of the structure in rock engineering analysis. It is important to improve the accuracy of the contact processing.

Firstly, a lost case of vertex-to-vertex contacts in O-DDA is discovered. To enhance the contact detection effectiveness and accuracy, a new improved algorithm that can overcome the lost case of vertex-to-vertex contacts in the O-DDA is proposed. This study is organized as follows. First, the angle-based contact method for 2D-DDA is briefly introduced. A lost case of contact processing in the O-DDA, which leads to unreasonable results for the specific blocks in the numerical simulation has been discovered using a new improved rule based on the contact vector method for the contact detection algorithm, thereby solving the contact indeterminacy issue. Finally, a series of examples were studied to verify the effectiveness and accuracy of the I-DDA. Apart from addressing the indeterminacy of vertex-to-vertex contact, the I-DDA algorithm also remedies the defect of the lost contact case in the O-DDA. With the new algorithm for contact processing, the

I-DDA can be more efficiently and accurately applied for both stable and dynamic numerical analysis.

Meanwhile, compared with the Finite element method (FEM), the DDA method has a significant advantage in dealing with discontinuous problems more efficiently as a kind of powerful discrete element methods (Shi, 1988; Lin et al., 1996; Sitar et al., 2005; Kamai et al., 2008; Do et al., 2017, 2020a, 2020b). However, the failure behavior of a discontinuous blocky system is mainly governed by the strength of the interface joints. The failure model of the interface joints in the original DDA plays an essential role in the simulation accuracy of the block system. Compared with FEM, the failure model used in DDA blocky system theory is simple. Two different criteria for contact laws on normal and shear direction are employed in the original two-dimensional DDA method. The maximum normal stress criterion in the normal direction and Mohr-Coulomb criterion in the shear direction, are used respectively for tensile failure and shear failure. The joint will yield once one of those two constitutive models that the stress states reach the critical state of failure. To improve the accuracy of DDA simulation, a great number of researchers focused on improvements of the DDA contact constitutive model (Yeung and Leong., 1997; Tsesarsky and Hatzor, 2006; Ma et al., 2011, 2017; Wu et al., 2005, 2007, 2009, 2010, 2011, 2017; Zhang et al., 2013, 2014, 2015). For example, Wang et al., (2013) developed the original DDA with displacement-dependent interface shear strength, which proved that adopting relative displacement between contacted blocks as the criterion for the removal of cohesion will provide more accurate results for landslide failure simulations. Song et al., (2016) considered the friction degradation of the sliding surface during landslide movement and incorporated a velocity and displacement dependent friction law into discontinuous deformation analysis (DDA) for numerical modeling of high-speed and long run-out landslides. Ma (2017a) implements the Barton-Bandis rock joint model in 2D DDA and provides an alternative approach to compute the interfacial shear strength of rock joints in 2D DDA. With the modified DDA model, Ma (2017b) further proved that the modified DDA model is capable of describing the different shear behaviors of rock joints subjected to cyclic loading conditions.

[Ibanez et al., \(2018\)](#) implemented a friction degradation law into DDA to model the deformation of the landslide with emphasis on sliding velocity, run-out distance, and event duration.

While most of their works focused on the improvement of the shear behavior of joint rock models in 2D DDA without consideration of tensile behavior, practically the tensile failure is also a significant behavior of rock failure. For earthquake-induced landslides, the shear failure is not the only factor relating to slope stability, and tensile failure can also be observed in many slope failure problems. For rock fragmentation analysis, the tension is the main factor controlling the rock failure. Tension behavior of the joint rock model in DDA is as crucial as shear behavior in simulating rock failure accurately. More intention should be paid on the tensile model in 2D DDA. However, few studies on constitutive models for tensile behaviors of rock joints in DDA have been reported. To simulating rock fragmentation in DDA, [Jiao \(2012\)](#) imposed two-phase force-displacement relation in the normal direction of the block interface instead of the penalty contacting spring to simulate the fragmentation of jointed rock. [Ning \(2011\)](#) presented the advanced discretization approach for fracturing modeling and discussed the influence of discretization strategy on the crack evolution paths. The final failure patterns of their simulated results agree favorably with those obtained from physical tests, which suggests that the DDA can be a suitable method for rock fragmentation problems.

In this study, to improve the calculation performance and accuracy of the two-dimensional DDA method in simulating tensile behaviors of materials, a new unified tensile fracture model is then incorporated into the DDA method. The unified tensile criterion is used for simulating the tensile failure of various materials. The accuracy of the I-DDA method for analyzing material's tensile behaviors is validated by simulating the direct tensile test and the indirect Brazil disk split tests. The simulated results are compared with those from experiment tests and other numerical analysis. Additionally, to investigate the failure models of the slope with different tensile strengths, a dynamic earthquake-induced landslide is analyzed by

using the I-DDA method. The failure model of the slope with the same shear strength but different tensile strength is presented.

3.2. TYPES OF CONTACT IN 2D DDA

As described in the previous section 2.2, contact processing is an important factor for accurately predicting the behavior of jointed rock mass, which is crucial for its application in safe and economical construction in rock engineering. In the application of solving complex engineering problems, such as landslide initiation and evolution (Sitar et al., 2005; Song et al., 2016, 2017; Wu et al., 2007, 2009, 2011; Chen et al., 2021), tunnel stability analysis (Wu et al., 2004; Tsesarsky and Hatzor, 2006; Kamai et al., 2008; Fu et al., 2016; Do et al., 2020a, 2020b), and rockfall analysis (Cheng et al., 2000; Chen et al., 1997, 2003, 2013; Ma et al., 2011; Wu et al., 2005;), there are complex block interactions involving a large number of blocks and arbitrary contact states. Accurately and effectively solving the contact processing in the DDA calculation is important for engineering analysis. In the original DDA method, three types of contacts were determined for two-dimensional DDA: vertex-to-vertex, vertex-to-edge, and edge-to-edge. As an edge-to-edge contact can be treated as two vertex-to-edge contacts, only two contact types (Figure 3.1), vertex-to-edge and vertex-to-vertex, were calculated in the DDA.

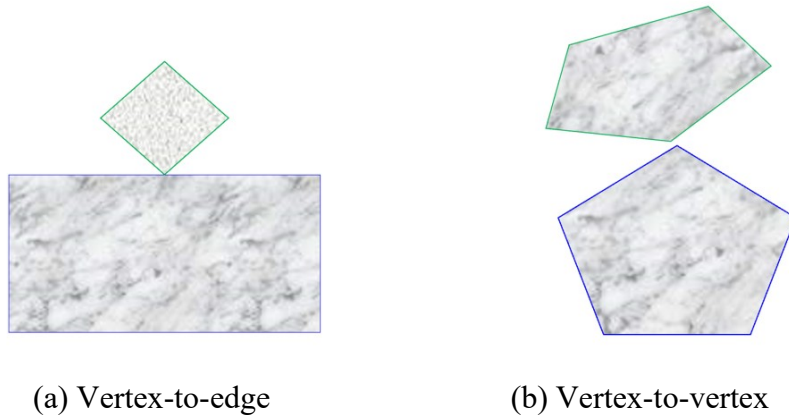


Figure 3.1 Contacts in DDA: (a) Vertex-to-edge; (b) Vertex-to-vertex

3.3. INTERACTION BETWEEN BLOCKS IN THE ORIGINAL DDA

The contact penalty springs are added or removed in both the normal and tangential direction of the boundary between the adjacent blocks to simulate the shearing and tensile failure of the block system. As presented in Figure 3.2, the normal spring is used to govern the tension or compression forces between the contact block, while the tangential spring is used to resist the shear force on the tangential direction between the contact blocks.

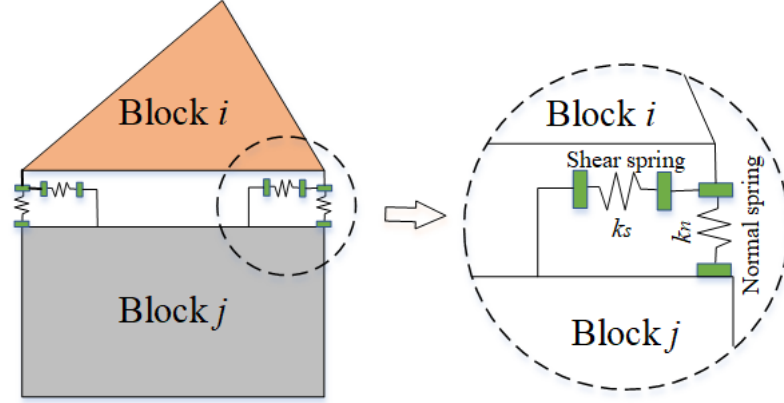


Figure 3.2 The normal and shear springs governing interaction between blocks the in DDA

The Mohr-Coulomb criterion is used in the original DDA to evaluate the shear failure of each contact. As shown in Figure 3.3, the vertex P_l of block i penetrates block j through point P_0 of interference edge with a depth of d_n on the normal direction and relative shear displacement d_s on the tangential direction. The resisting shear stress will occur because of the tangent penalty springs. When the shearing stress is smaller than the shear strength of the Mohr-Coulomb friction criterion, the two blocks will move and deform as a whole. While the shearing stress is larger than or equal to the critical shear stress of the Mohr-Coulomb criterion, the tangent penalty springs will be removed, and the shear failure occurs. The shear failure can be explained by the following equation:

$$k_s d_s \geq k_n d_n \tan \varphi + cl \quad (3.1)$$

where k_s is the stiffness of the shear springs, d_s is the relative shear displacement of the contact; k_n is the stiffness of the normal springs, d_n is the penetration in normal direction; φ is the joint friction; c is the cohesion of per unit length; l is the length of the contact.

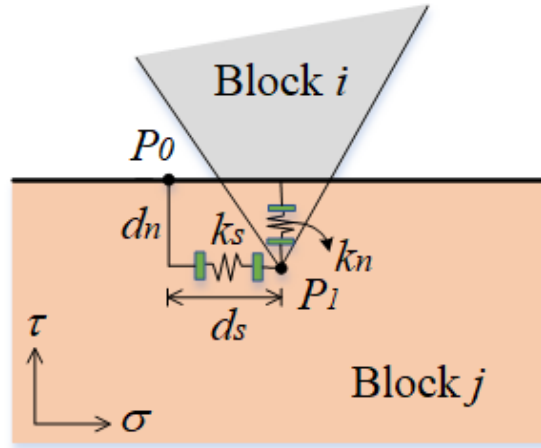


Figure 3.3 Determination of the shear and tensile failure in DDA

Meanwhile, the maximum normal stress criterion is used for the evaluation of the tensile failure of each contact. If the top block is acted with a tensile force P , the normal force induced by the normal springs to resist the tension P . When the tensile stress is less than the tensile strength, the two blocks will move and deform as a whole. When the tensile stress is larger than or equal to the tensile strength, the tensile failure occurs. In the original DDA, it can be expressed by:

$$k_n d_n \leq -Tl \quad (3.2)$$

where k_n is the stiffness of the normal springs, d_n is the penetration in normal direction; T is the tensile strength, and l is the length of the contact.

Two criteria are used respectively for the shear failure and tensile failure evaluation of each contact in the original DDA. According to the normal component and shear component of the contact force, the three contact states ‘open’ ‘sliding’ and ‘locked’ between the blocks are determined, which can be explained by:

$$k_n d_n > 0, \quad k_s d_s \leq k_n d_n \tan \varphi + cl \quad \rightarrow \text{locked} \quad (3.3)$$

$$k_n d_n > 0, \quad k_s d_s > k_n d_n \tan \varphi + cl \quad \rightarrow \text{sliding} \quad (3.4)$$

$$k_n d_n \leq -Tl \quad \rightarrow \text{open} \quad (3.5)$$

Notes that in DDA contact check once the contact state changes from the ‘locked’ to ‘open’ or ‘sliding’, the cohesion of each contact will be removed while the friction angle of the contact will remain the same for shearing failure procedure.

The original DDA works well to solve the engineering problems for rock with real joints. However, the joint model with two criteria has limitations for simulating the tensile behavior of various brittle materials, like rock, soil, and concrete, because of the simple tensile fracture model. Additionally, the normal stress and shear stress are both considered according to the Mohr-Coulomb friction law during the shear failure process. In contrast, only tensile stress is considered in the tensile fracture model, which is also unreasonable in physics when simulating the tensile behavior of material for which the standard max normal stress criterion is improper to use, such as intact rock. This paper attempts to incorporate a new unified tensile failure model, which is more suitable for solving the shear failure and tensile fracture problems of various engineering materials in the DDA method.

3.4. MODIFICATION OF VERTEX-TO-VERTEX CONTACT PROCESSING

In the vertex-to-edge contact, there is usually no indeterminacy during the calculation process because the determination of the contact reference line is clear and specific. For vertex-to-vertex contact, the determination of the contact reference line is complex and will lead to failure calculation if the parameters related to the contact detection are not appropriately chosen. In general, the contacts for the vertex-to-vertex are generalized into four possible contact cases based on the angle method in the O-DDA program. As shown in Figure 3.4, when the two blocks contact vertex-to-vertex, four reference lines (OE_1 , OE_2 , OE_3 , OE_4) and two angles (a and b) are proposed to determine the four possible contacts for contact processing in the O-DDA program.

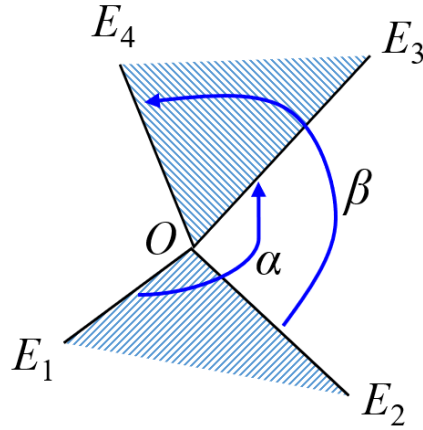


Figure 3.4 Contact processing for the vertex-to-vertex contact

The vertex-to-vertex contact can be classified into four possible contact states based on the angles a and b (Figure 3.4): (a) $a \leq 180^\circ$, $b \leq 180^\circ$, OE_2 and OE_3 are two reference lines; (b) $a \leq 180^\circ$, $b > 180^\circ$, OE_3 and OE_4 are two reference lines; (c) $a > 180^\circ$, $b \leq 180^\circ$, OE_1 and OE_2 are two reference lines; (d) $a > 180^\circ$, $b > 180^\circ$, OE_1 and OE_4 are two reference lines. Figure 3.5 shows the different reference lines (thick red line) for four possible vertex-to-vertex contacts in the O-DDA program. If the two reference lines are passed by the corresponding vertices of the other angles simultaneously, two reference lines are chosen.

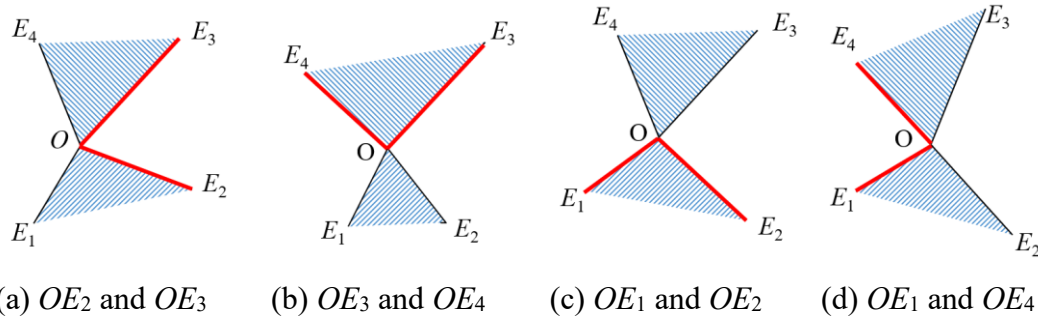
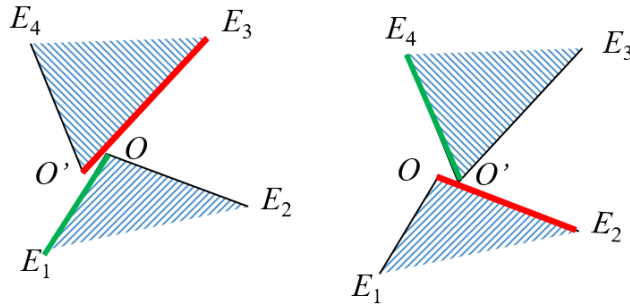


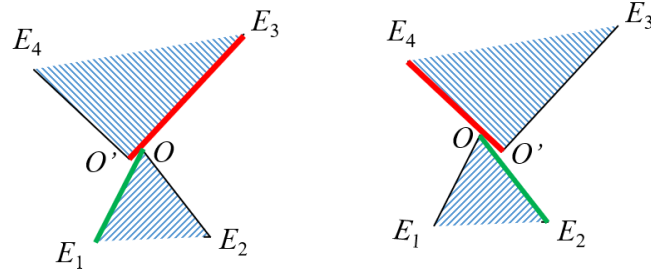
Figure 3.5 Possible reference lines of the four cases for the vertex-to-vertex contact

Technically, the real vertex-to-vertex contact is defined as the point-to-point contact of the two corners in the block system. In theory, the distance between the two vertices of corners is zero. In the O-DDA, a threshold value of distance d_0 , which is twice the maximum displacement r of points of all blocks ($d_0 = 2\rho$), is

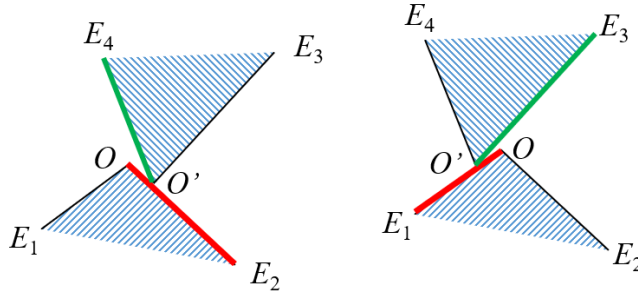
defined for contact detection. If the distance of two points d_v is greater than a threshold value of the distance d_0 , then it is impossible for the block I and block J to meet during the next step. suppose that the distance between two vertices d_v is smaller than a certain threshold value, $d_v \leq 2\rho$, the two vertices can be considered as quasi-vertex-vertex contact, which is a candidate for vertex-to-vertex contact and will eventually convert into vertex-to-edge contact. The reference line will be different with different positions of the vertex-to-vertex contact, even though the angles a and b are the same. As shown in Figure 3.6, there are two different situations according to the difference in the location of the two corners for each vertex-to-vertex contact case. For example, as shown in Figure 3.6 (a), if the angles $a \leq 180^\circ$ and $b \leq 180^\circ$, there are two pairs of quasi-vertex-vertex contact; i.e., the reference line may be the edges $O'E_3$ or OE_2 . In O-DDA, the distance method is then adopted for quasi-vertex-vertex contact, and the reference line can be determined based on the distance between the vertex and the candidate reference line. However, as shown in Figure 3.6, the thick green line that can possible be the reference line of the other angles, is different for each case, and has not been considered in the O-DDA. If the possible reference lines are not properly determined, it will lead to indeterminacy when using the distance method to determine the right reference in the next step.



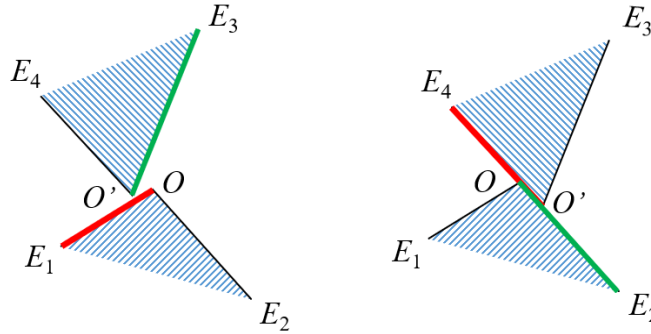
(a): $a \leq 180^\circ, b \leq 180$



(b) $a \leq 180^\circ, b > 180^\circ$



(c) $a > 180^\circ, b \leq 180^\circ$



(d) $a > 180^\circ, b > 180^\circ$

Figure 3.6 Reference lines of the four cases: Case (a): $\alpha \leq 180^\circ, \beta \leq 180^\circ$; Case (b) $\alpha \leq 180^\circ, \beta > 180^\circ$; Case (c) $\alpha > 180^\circ, \beta \leq 180^\circ$; Case (d) $\alpha > 180^\circ, \beta > 180^\circ$

If two blocks are embedded into each other, the method with the shortest path will be used to select the reference line for the quasi-vertex-vertex contact in those states. The reference line can be determined by comparing the normal penetrating distances between the reference lines and the corresponding point. A contact spring is attached between the point and its reference line to push the invaded angle out of the block along the shortest path. However, there is no special treatment in the O-

DDA for the case with two equal penetration distances. Therefore, the O-DDA method has limitations and indeterminacy for vertex-to-vertex contact.

3.4.1. A LOST CASE OF CONTACT DETECTION IN THE ORIGINAL DDA

The method for determining the reference line in the O-DDA can partially solve vertex-to-vertex contact problems, but there are some limitations. The original Shi's method for contact detection may lead to an intermediate state in some individual cases, particularly for contact blocks with arbitrary geometry. In this study, a lost case of the vertex-to-vertex contact was discovered in the O-DDA program. Figure 3.7 shows the contact state of the normal case and the lost case for determining the candidate reference line. As shown in Figure 3.7 (a), the two blocks are detected as vertex-to-vertex contact at points P_i and P_j . The edges $P_{j+1}P_j$ and P_iP_{i-1} are detected as candidate reference lines for the vertices P_i and P_j , respectively, as the angles $a \leq 180^\circ$ and $b \leq 180^\circ$. However, if two blocks collide with the vertex-to-vertex contact position in Figure 3.7 (b), the angles a and b remain unchanged in this case. Based on the angle-based contact method in the O-DDA, the candidate reference lines should be $P_{j+1}P_j$ and P_iP_{i-1} , which is the same as the reference lines in Figure 3.7 (a). The procedure for detecting the reference line might be inaccurate in this situation because the candidate reference line for vertex P_i , in theory, should be P_jP_{j-1} , as shown in Figure 3.7 (c), rather than P_jP_{j+1} . This will lead to inaccuracies in predicting the behavior of simulated blocks.

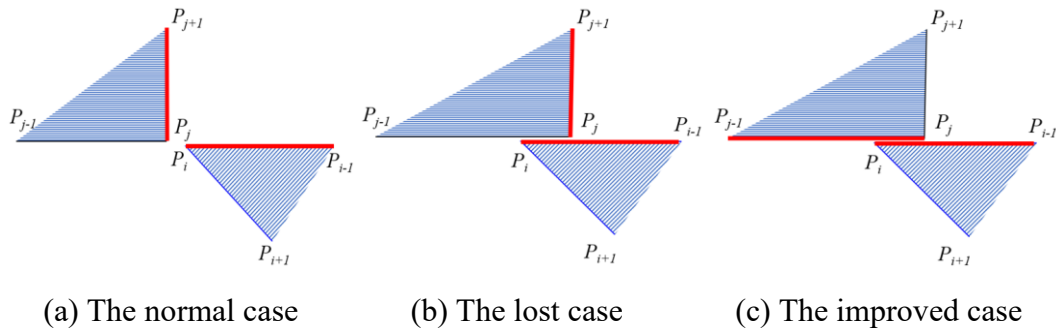


Figure 3.7 A lost case of contact detection: (a) The normal case; (b) The lost case; (c) The improved case

If the lost contact case cannot be solved effectively, it will lead to inaccurate results in the numerical analysis using the O-DDA method owing to the

indeterminacy of detecting the right reference line. Consequently, the contact spring will be added incorrectly as the reference line was detected inappropriately. This will significantly affect the behavior of the simulated blocks in the numerical analysis. As shown in Figure 3.8, a simple block model was built to study the effect of contact processing on numerical seismic response analysis. The left block is subjected to a seismic wave with a peak ground acceleration (PGA) of 11.56 m/s^2 in the horizontal direction, as shown in Figure 3.9. A series of smaller right square blocks of the same size were bonded to the base block with joint contacts.

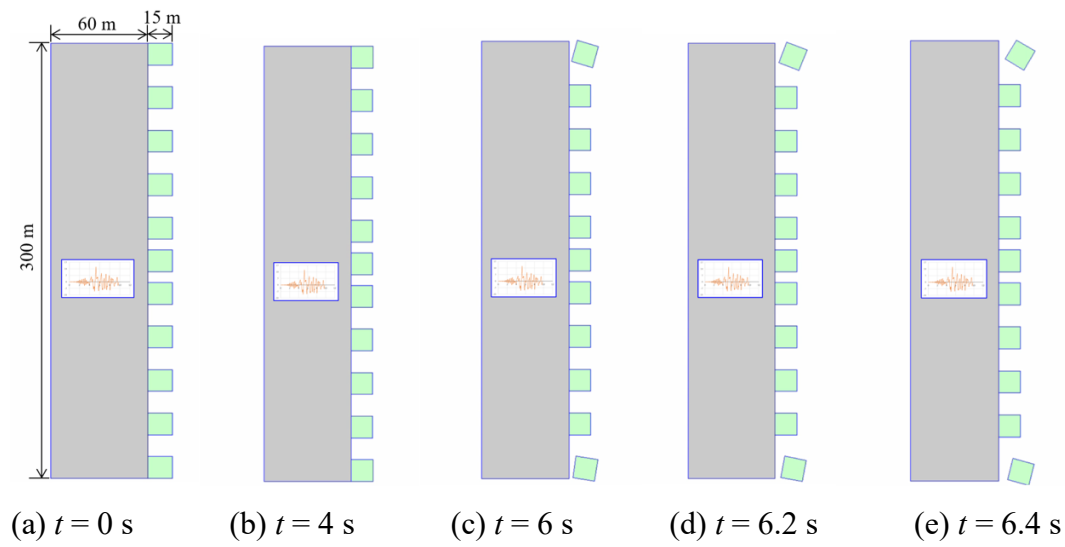


Figure 3.8 Numerical seismic response analysis of O-DDA program

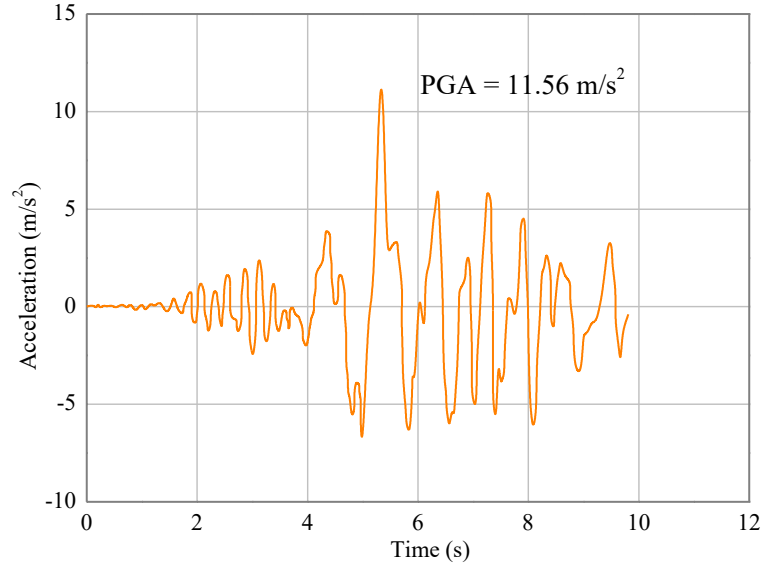


Figure 3.9 Seismic wave for response analysis

The material properties of the blocks are as follows: mass density $\rho = 2500 \text{ kg/m}^3$, Young's modulus $E = 10 \text{ GPa}$, and Poisson's ratio $\nu = 0.02$. The stiffness of the normal contact spring k_n is 0.1 GPa , the maximum displacement ratio R is 0.01 , and the time increment for each time step is 0.01 s . The joint parameters for the contact interface between the blocks were as follows: $c = 3 \text{ MPa}$, and $\varphi = 0^\circ$. The total number of computation steps $N = 800$ was executed. The gravity of the blocks was also considered in this case study.

Figure 3.8 shows the seismic response analysis results obtained by the O-DDA in which contact detection is conducted based on the angle method. Although the same strength parameters were set for all blocks, the two blocks at the top and bottom were separated from the left block at first during the seismic response analysis. In contrast, the other blocks remained stable with the left block in the entire seismic response analysis. The unstable contact between these blocks led to the separation of the top and bottom two blocks. For these two separated blocks, the vertex-to-vertex contact in the previously proposed lost case was detected in this case. Thus, the reference lines cannot be determined accurately. As a result, the contact spring cannot be added properly, resulting in the separation of the contacts between the top and bottom blocks in the seismic response analysis.

3.4.2. A NEW CRITICAL CONCEPT FOR CONTACT DETECTION

It can be seen that the angle method for contact detection in O-DDA will lead to inaccurate results under a specified condition and will detect incorrect reference lines for the vertex-to-vertex contact in the lost case proposed previously. This will further lead to unreasonable simulation results in the O-DDA method without proper reference lines. To remedy this defect and to detect the reference lines for the vertex-to-vertex contact accurately, a new improved concept is proposed for contact detection in DDA.

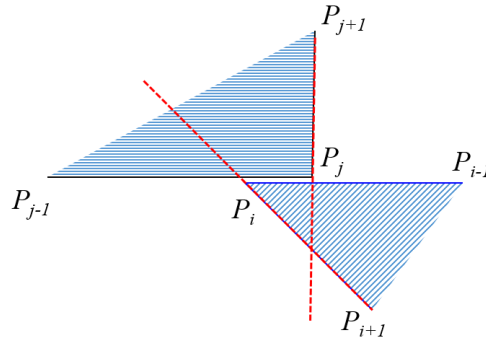


Figure 3.10 New critical concept for the vertex-to-vertex contact: inside or outside of a reference line

The new improved concept for detecting the reference lines of the vertex-to-vertex contact consists of two cases: inside or outside of a reference line, which is a stricter and more effective algorithm for contact detection in I-DDA. If a vertex is detected to be inside a reference line, it can be determined that the inside reference line cannot be the candidate entrance line for this vertex. If a vertex is determined outside a reference line, this line can be regarded as a candidate entrance line for this vertex. The cross product of the vectors is calculated to determine whether the vertex is inside or outside the reference line. To be specific, as shown in Figure 3.10, for any two blocks detected to have vertex-to-vertex contact with vertices P_i and P_j , the cross product of vectors is calculated to determine the reference line for the vertex P_i and P_j , and expressed by the following equations:

$$\overrightarrow{P_i P_{j+1}} \times \overrightarrow{P_i P_j} < 0, \text{ and} \quad (3.6)$$

$$\overrightarrow{P_j P_{i+1}} \times \overrightarrow{P_j P_i} < 0 \quad (3.7)$$

where both the cross products of the vectors in Eq. (3.6) and (3.7) are negative, which means that the vertex P_i is inside the reference line P_jP_{j+1} , and the vertex P_j is inside the reference line P_iP_{i+1} . Therefore, the line P_jP_{j+1} cannot be determined as the reference line of P_i , and the line P_iP_{i+1} cannot be the reference line of P_j .

Meanwhile, the other two vectors are calculated for contact detection of vertex P_i and P_j , which can be expressed as follows:

$$\overrightarrow{P_iP_{j+1}} \times \overrightarrow{P_iP_j} \geq 0, \text{ and} \quad (3.8)$$

$$\overrightarrow{P_jP_{i+1}} \times \overrightarrow{P_jP_i} \geq 0 \quad (3.9)$$

where both the cross products of the vectors in Eq. (3.8) and (3.9) are greater than or equal to zero, which means that the vertex P_j is outside the reference line $P_{i+1}P_i$ and the vertex P_i is outside the reference line $P_{j+1}P_j$. Thus, the lines $P_{i+1}P_i$ and $P_{j+1}P_j$ are determined as the reference lines of vertex P_j and vertex P_i , respectively.

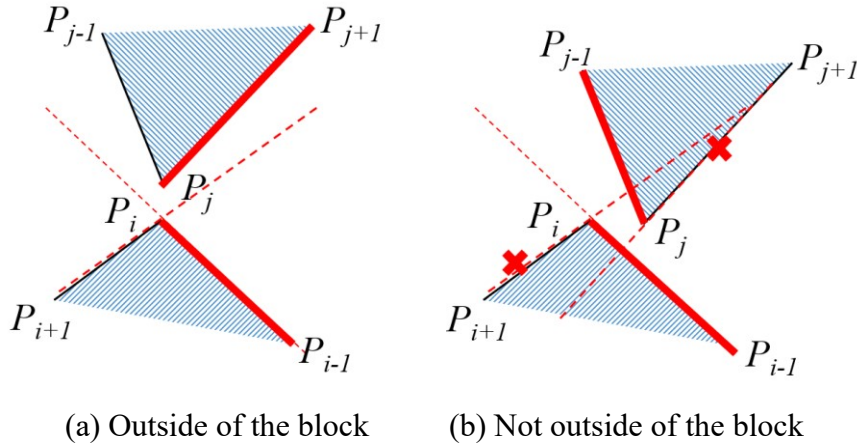


Figure 3.11 Outside or not outside of the block: (a) Outside of the block; (b) Not outside of the block

Based on the cross-products of vectors for two points in contact, they can be categorized into different cases of vertex-to-vertex contact. As shown in Figure 3.11 (a), we can see that vertex P_j is outside of both reference lines $P_{i+1}P_i$ and P_iP_{i-1} because the cross products $\overrightarrow{P_jP_{i+1}} \times \overrightarrow{P_jP_i} \geq 0$ and $\overrightarrow{P_jP_i} \times \overrightarrow{P_jP_{i-1}} \geq 0$. Thus, the vertex P_j is regarded as outside block I . Meanwhile, the vertex P_i is outside both the reference lines $P_{j+1}P_j$ and P_jP_{j-1} as the cross products $\overrightarrow{P_iP_{j+1}} \times \overrightarrow{P_iP_j} \geq 0$ and $\overrightarrow{P_iP_j} \times \overrightarrow{P_iP_{j-1}} \geq 0$. Hence, the vertex P_i is considered to be outside block J . In this situation, both

vertices are outside the contact block, which is the normal case in contact detection. The original angle method is valid for detecting the reference line in this case. According to the original Shi's method (Figure 3.6), the lines $P_{j+1}P_j$ and P_iP_{i-1} should be detected as the reference lines for the vertices P_i and P_j . Otherwise, as shown in Figure 3.11 (b), when a vertex P is outside one edge and inside another line, the new concept for detecting the reference line can be classified as follows:

- (1) The vertex P_i is outside the line P_jP_{j-1} and inside the line $P_{j+1}P_j$. Thus, $P_{j+1}P_j$ cannot be a reference line, and P_jP_{j-1} is detected as the reference line of vertex P_i .

$$\overrightarrow{P_iP_{j+1}} \times \overrightarrow{P_iP_j} < 0 \quad (3.10)$$

$$\overrightarrow{P_iP_j} \times \overrightarrow{P_iP_{j-1}} \geq 0 \quad (3.11)$$

- (2) The vertex P_i is outside the line $P_{j+1}P_j$ and inside the line P_jP_{j-1} . Thus, P_jP_{j-1} cannot be a reference line, and $P_{j+1}P_j$ is detected as the reference line of vertex P_i .

$$\overrightarrow{P_iP_{j+1}} \times \overrightarrow{P_iP_j} \geq 0 \quad (3.12)$$

$$\overrightarrow{P_iP_j} \times \overrightarrow{P_iP_{j-1}} < 0 \quad (3.13)$$

- (3) The vertex P_j is outside the line P_iP_{i-1} and inside the line $P_{i+1}P_i$. Thus, $P_{i+1}P_i$ cannot be a reference line, and P_iP_{i-1} is detected as the reference line of vertex P_j .

$$\overrightarrow{P_jP_{i+1}} \times \overrightarrow{P_jP_i} < 0 \quad (3.14)$$

$$\overrightarrow{P_jP_i} \times \overrightarrow{P_jP_{i-1}} \geq 0 \quad (3.15)$$

- (4) The vertex P_j is outside the line $P_{i+1}P_i$ and inside the line P_iP_{i-1} . Thus, P_iP_{i-1} cannot be a reference line, and $P_{i+1}P_i$ is detected as the reference line of vertex P_j .

$$\overrightarrow{P_jP_{i+1}} \times \overrightarrow{P_jP_i} \geq 0 \quad (3.16)$$

$$\overrightarrow{P_jP_i} \times \overrightarrow{P_jP_{i-1}} < 0 \quad (3.17)$$

- (5) The vertex P_i is inside the block J .

$$\overrightarrow{P_iP_{j+1}} \times \overrightarrow{P_iP_j} < 0 \quad (3.18)$$

$$\overrightarrow{P_i P_j} \times \overrightarrow{P_i P_{j-1}} < 0 \quad (3.19)$$

(6) The vertex P_j is inside the block I .

$$\overrightarrow{P_j P_{i+1}} \times \overrightarrow{P_j P_i} < 0 \quad (3.20)$$

$$\overrightarrow{P_j P_i} \times \overrightarrow{P_j P_{i-1}} < 0 \quad (3.21)$$

The two blocks are embedded into each other if the vertex has been detected to be inside the block, as expressed in cases (5) and (6). In this case, the original Shi method will be applied to select the reference line instead of the proposed method. However, an enhancement to the shortest path method was proposed for the case when the two penetration distances were equal. In this case, the two penetration lines were selected as the reference lines simultaneously. The new concept is mainly an enhancement of contact when there is no penetration between two blocks for the vertex-to-vertex contact. The new algorithm for detecting the reference line of the vertex-to-vertex contacts remedies the discovered lost case in the O-DDA. When the vertex P is inside one reference line, the reference line should be changed to the other edge, where P is determined to be outside. When the vertex is outside or inside both the possible edges, the original rules are valid. Figure 3.12 shows a diagram of the new concept of detecting the reference line for vertex-to-vertex contact in the I-DDA program.

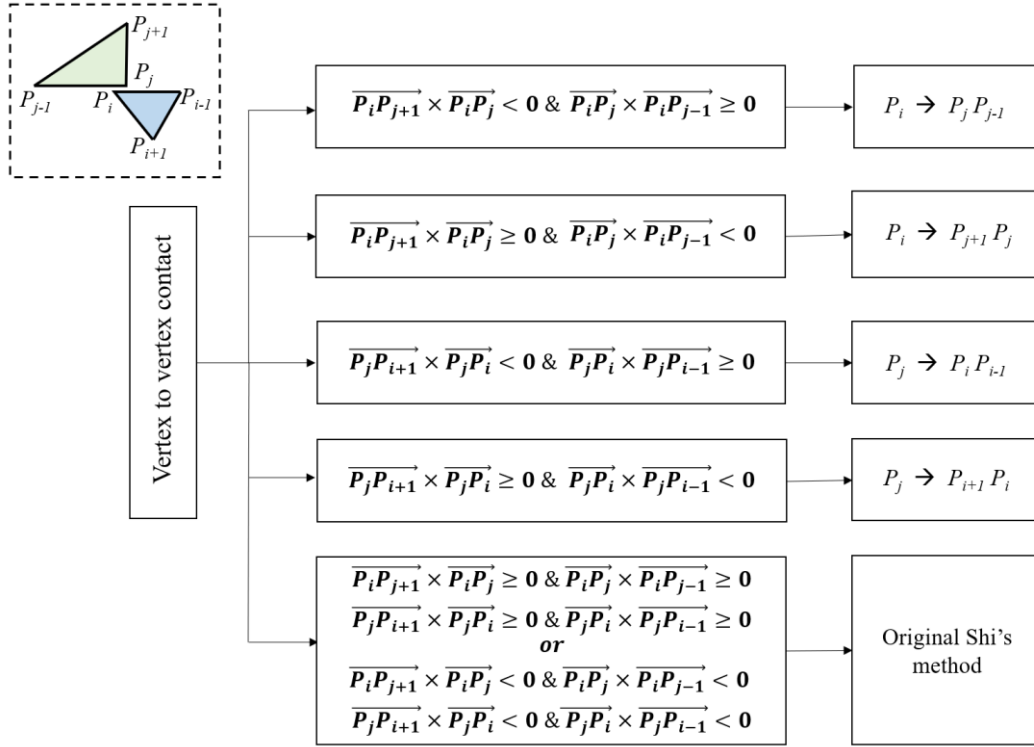


Figure 3.12 Diagram of the reference line detection of the new concept for vertex-to-vertex contact in I-DDA

3.4.3. VALIDATION EXAMPLES

3.4.3.1. STABILITY VALIDATION OF BLOCK CONTACT UNDER THE SEISMIC WAVE

As described previously, a numerical seismic response analysis with the block model in Figure 3.8 was conducted to validate the effectiveness of the I-DDA with the new improved contact detection concept. The material properties for all blocks and the seismic waves were the same as those in the previous simulation.

As shown in Figure 3.13, it can be seen that both the top and bottom blocks remain stable as other blocks under the seismic wave, suggesting that the lost case of the vertex-to-vertex contact in the O-DDA can be solved using the I-DDA method. I-DDA can be effectively applied to seismic response analyses.

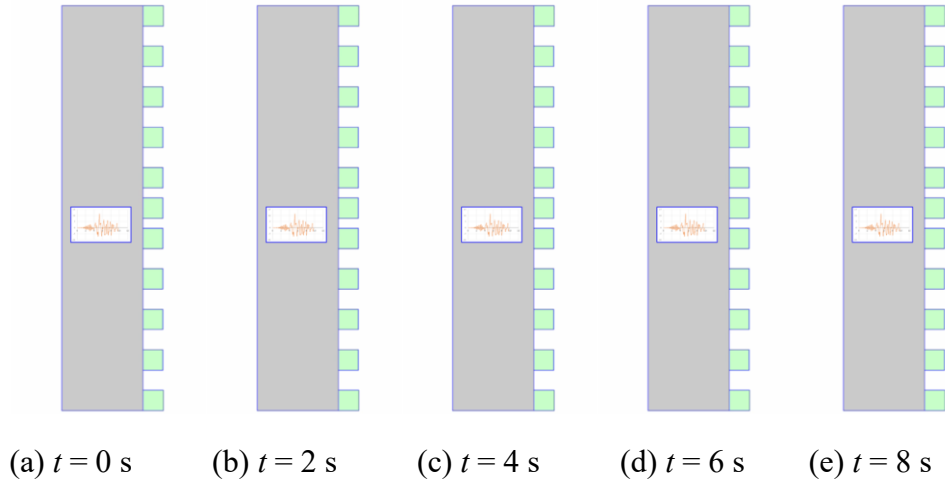


Figure 3.13 Results of the I-DDA program

The separation of the small square block can be explained by inaccurate contact detection in the O-DDA. As shown in Figure 3.14, the vertices O and O' are detected as a vertex-to-vertex contact in this case because the distance between the vertices O and O' is small, which is less than the distance threshold value. Based on the angle method for contact detection in the O-DDA, the lines $O'E_2$ and OE_3 will be detected as the reference lines for the vertices O and O' , which is incorrect in this case. Then, the contact springs between the two blocks cannot be accurately added. As a result, the block easily loses stability under seismic waves.

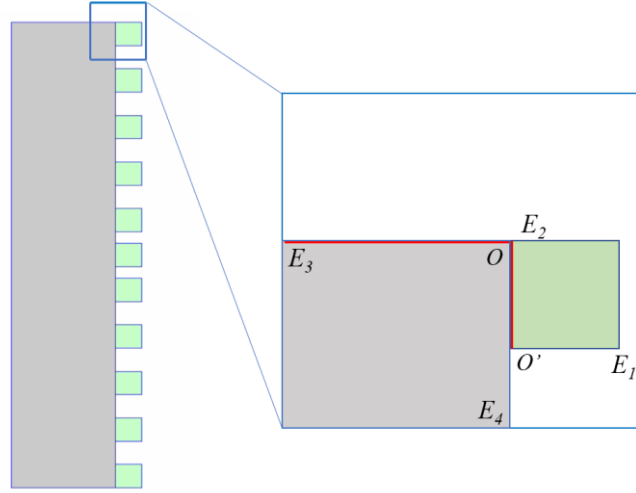


Figure 3.14 Reference line detection in the O-DDA

In the I-DDA with the new contact detection algorithm, the lines P_jP_{j-1} and P_iP_{i-1} are detected as the reference lines for the vertices P_i and P_j , as shown in Figure 3.15. The contact springs were set accurately between the contact interfaces of the blocks. Thus, the top and bottom blocks can remain as stable as the other blocks under the seismic wave.

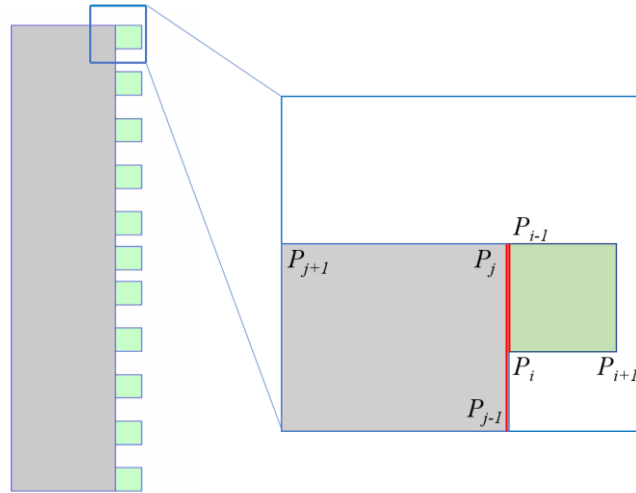


Figure 3.15 The reference line detection in the I-DDA

3.4.3.2. A SLIDING BLOCK MODEL

The sliding block model is a useful method that has been used to validate the accuracy of the DDA model accuracy in previous studies (Ni et al., 2020a, 2020b). To verify the effectiveness and accuracy of the I-DDA with the newly proposed

contact processing method, a sliding block model was also simulated in this study. As shown in Figure 3.16, five fixed blocks were bonded together as an incline with a slope angle of 45° . A block with different friction angles ($\varphi = 30^\circ, 20^\circ$, and 10°) slides under gravity with zero initial velocity on the slope. A measured point was used on this sliding block to record the sliding displacement.

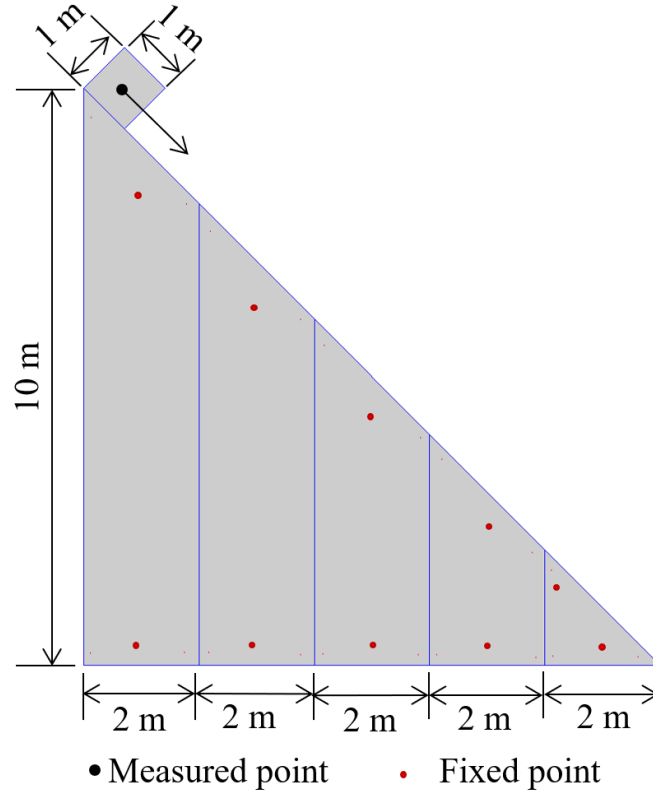


Figure 3.16 Sliding block model

Figure 3.17 shows the sliding block's analytical and simulated displacements with the friction angle of $30^\circ, 20^\circ$, and 10° , respectively, within 2s. When the upper block slides down, the edge-to-edge, vertex-to-edge, and vertex-to-vertex contact types get frequently involved. As seen from Fig. 15, the analytical and I-DDA-simulated sliding displacement are concurrent, proving the accuracy of the I-DDA with the new contact detection method.

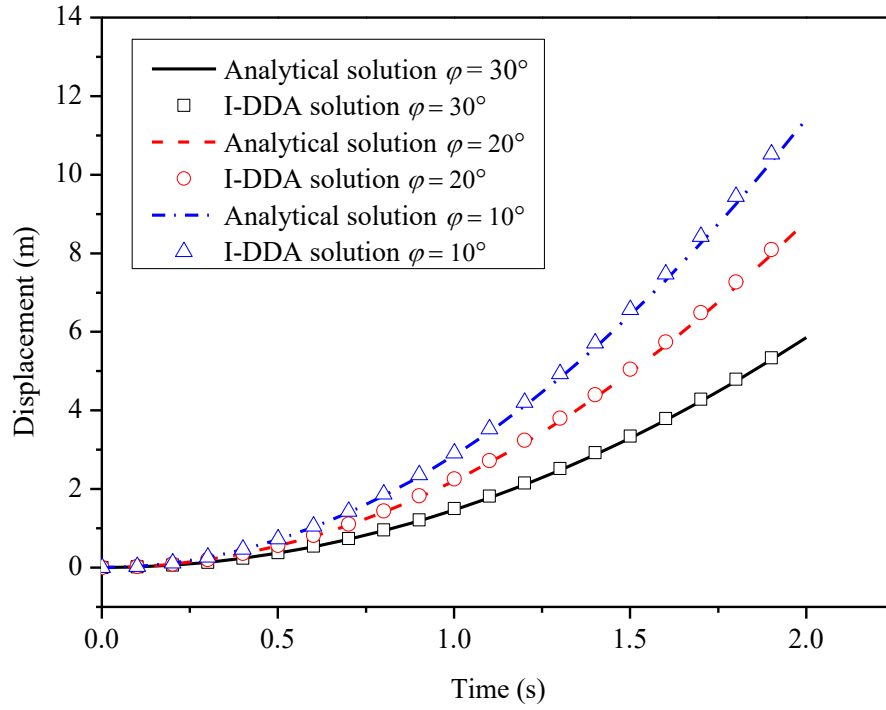


Figure 3.17 Results of a sliding block model

3.4.3.3. VERTEX-TO-VERTEX CONTACT WITH THE CONVEX ANGLE

Figure 3.18 shows a block system with two blocks: the top and bottom blocks and contact with convex angles. Under gravity, the top block falls and collides with the bottom fixed block at the vertex-to-vertex contact state. For comparison, the model was calculated using both the original and the I-DDA programs.

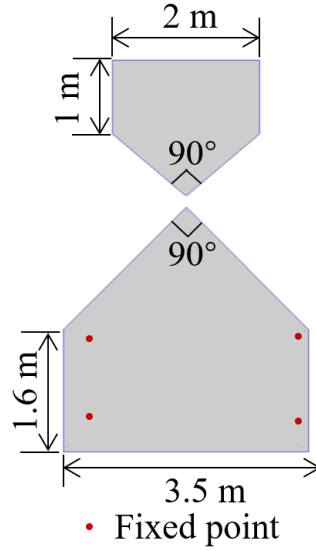


Figure 3.18 Model of the vertex-to-vertex contact with the convex angle

The material properties for the two blocks are as follows: mass density $\rho = 2500 \text{ kg/m}^3$, Young's modulus $E = 10 \text{ GPa}$, and Poisson's ratio $\nu = 0.02$. The stiffness of the normal contact spring is set automatically by using an interactive visualization system for DDA to exclude the effect of the parameter setting in the simulation (Chen et al., 1997). Normally, the value of spring stiffness is set from $10 E$ to $1000 E$ to make sure that the displacement of the spring is 10^{-3} to 10^{-4} times the total displacement. The allowed maximum displacement ratio is $R = 0.01$, and the time increment for each time step is 0.01 s .

Figure 3.19 shows the results obtained from the O-DDA. It can be observed that the top block first fell under gravity and was vertically rebounded with larger velocity after the collision with the bottom block at the right vertex-to-vertex contact. Then, the top block fell again and collided with the bottom block with a larger velocity. After several such collisions, the contact force between the two blocks became increasingly higher, and the two blocks collided with an unusually large velocity. Finally, the top block penetrated the bottom block owing to the exorbitant velocity. The result in the O-DDA is unreasonable because of inaccurate contact processing in vertex-to-vertex contact.

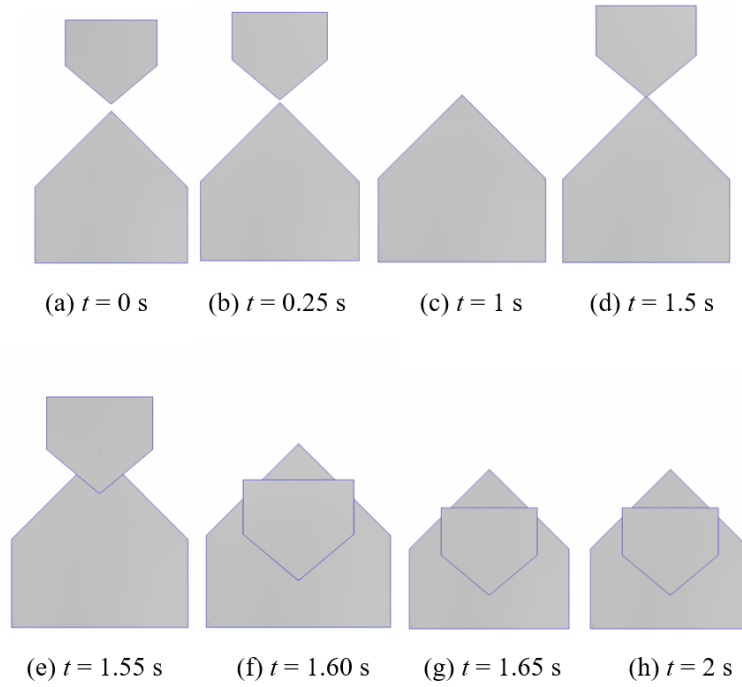


Figure 3.19 Results of the O-DDA program

In the I-DDA, as shown in

Figure 3.20, the top block falls and collides with a convex corner of the bottom block at the right vertex to the vertex point. It is then rebounded vertically with a low velocity. After several collisions, the top block gradually came to a standstill and remained stable on the bottom block. The results obtained by I-DDA are more reasonable and practical compared with the results from the O-DDA. The reason for the different results can be explained as follows: In the O-DDA, the shortest path method is applied to push the invaded angle out of the block along the shortest path between the vertex and the reference line. However, there is no special treatment in the O-DDA for the case with two equal penetration distances. Therefore, to overcome the limitation of the shortest path method, two potential reference lines of a vertex are selected for the case when the two penetration distances are equal in the I-DDA. Thus, the results of the O-DDA will be abnormal because there is indeterminacy in determining which edge is the reference line. As stated in the O-DDA, only one stiff spring is applied for one invaded angle in this vertex-to-vertex contact, and the spring stiffness is large enough to push the invaded angle out. Because the stiffness is automatically taken by the program, it will be

changed step-by-step in the simulation and increases as the number of block collisions and the penetration distance increases. Therefore, the velocity of the top block becomes increasingly higher in the original DDA. In the I-DDA, the two edges are selected as the reference lines and two stiff springs are applied simultaneously for each invaded angle of the contact. The normal invaded angle will be pushed out along the path in the vertical direction with springs of smaller stiffness.

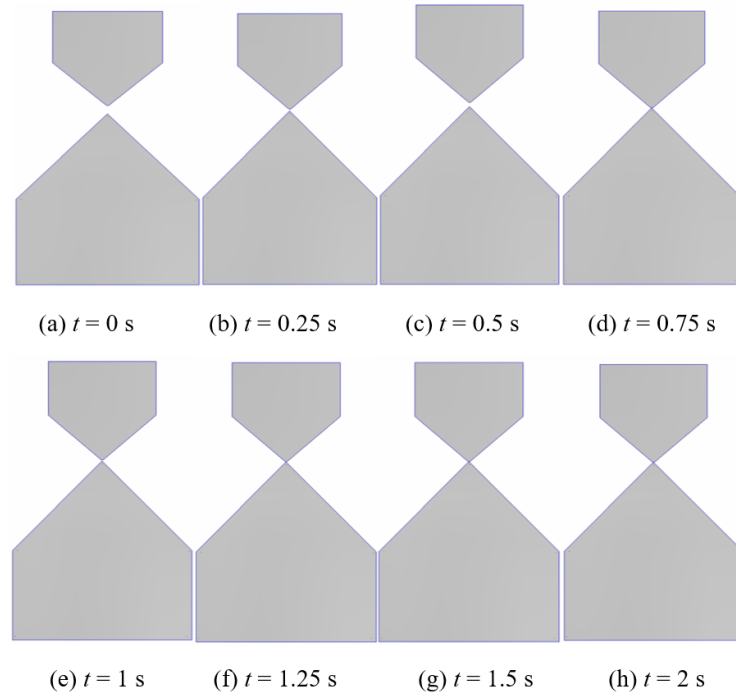


Figure 3.20 Results of the I-DDA program

3.4.3.4. VERTEX-TO-VERTEX CONTACT WITH THE CONCAVE ANGLE

As shown in Figure 3.21, the block system consists of one block with a convex corner and one block with a concave corner. The top block falls under gravity and comes into contact with the bottom block at the appropriate point of the concave corner. The model was calculated using both the original and I-DDA programs. The material properties and control parameters in this simulation are the same as those in the previous model shown in Figure 3.18.

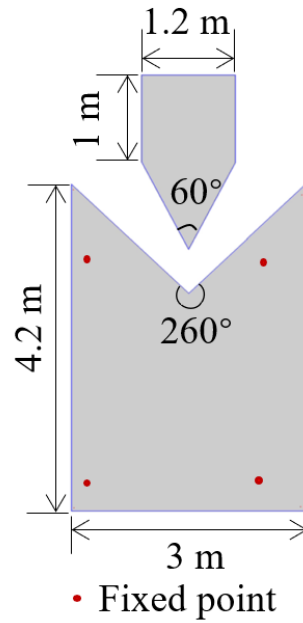


Figure 3.21 Model of the vertex-to-vertex contact with the concave angle

In the O-DDA, an abnormal simulation result was observed, as shown in Figure 3.22. The top block first falls under gravity and is rebounded vertically with a large interaction force. Then, the top block falls again and is rebounded with a higher velocity. After several such collisions, the top block runs out of the block system at a high velocity. The calculation in the O-DDA is unreasonable because of the incorrect contact processing between the two blocks.

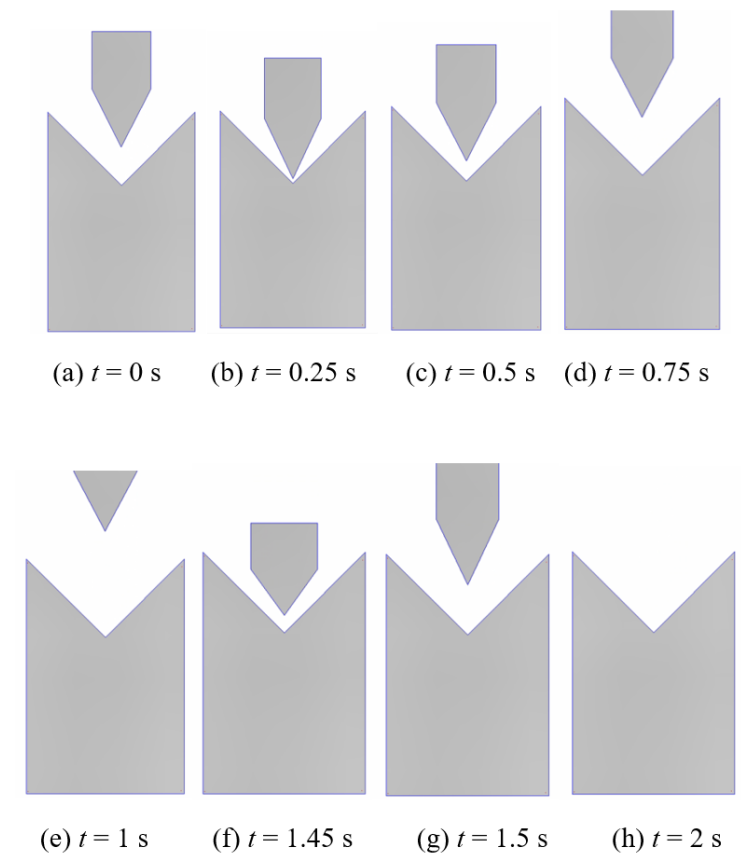


Figure 3.22 Results of the O-DDA program

As shown in Figure 3.23, the simulated results of this model were significantly improved in the I-DDA. The top block falls and collides at the vertex point of the concave corner of the bottom block. Then, it is rebounded vertically with a low velocity. After several collisions, the top block gradually slows down and remains stable on the bottom block. The same reason in the case study in Section 5.3 leads to the different results for O-DDA and I-DDA.

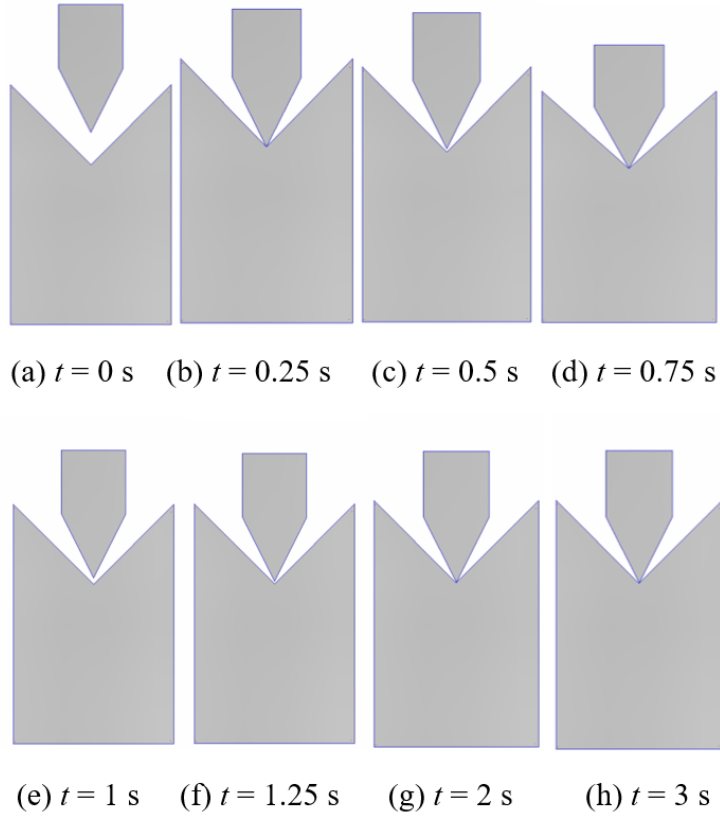


Figure 3.23 Results of the I-DDA program

3.5. INCORPORATING A UNIFIED TENSILE CRITERION

3.5.1. A UNIFIED TENSILE CRITERION

The classical failure criteria, i.e., maximum normal stress criterion, Tresca criterion, Mohr-Coulomb criterion, and von Mises criterion, are widely used for tensile failure modes. However, it is hard to compare which one is more suitable for various materials, thus a unified tensile fracture criterion to describe all the possible failure modes, named “ellipse criterion”, was proposed by [Zhang and Eckert \(2005\)](#). In this paper, the unified tensile fracture criterion was introduced in the DDA method to exemplarily describe the tensile fracture behavior of a variety of materials. The ellipse criterion can be represented by:

$$\frac{\sigma^2}{\sigma_0^2} + \frac{\tau^2}{\tau_0^2} \geq 1, (\sigma_0 > 0, \tau_0 > 0) \quad (3.22)$$

where σ is the normal stress; τ is the shear stress; σ_0 and τ_0 are the critical normal tensile and shear fracture stress of the material, respectively. The ratio $a = \tau_0 / \sigma_0$ ($a > 0$) is defined to reflect differences in the atomic structure or microstructure of different classes of materials. It can be seen that the tensile failure of this new criterion depends on both the normal stress σ and shear stress τ . It is worth mentioning that the Mohr-Coulomb criterion and the maximum normal stress criterion in the original DDA will be used when $\tau_0 = 0$ or $\sigma_0 = 0$.

As shown in Figure 3.24, each of the classical failure criteria can be expressed by the above ellipse criterion depending on the difference of the ratio a . The specified ellipse criteria with different values of α approximatively represent different classical failure criteria (see Table 3.1).

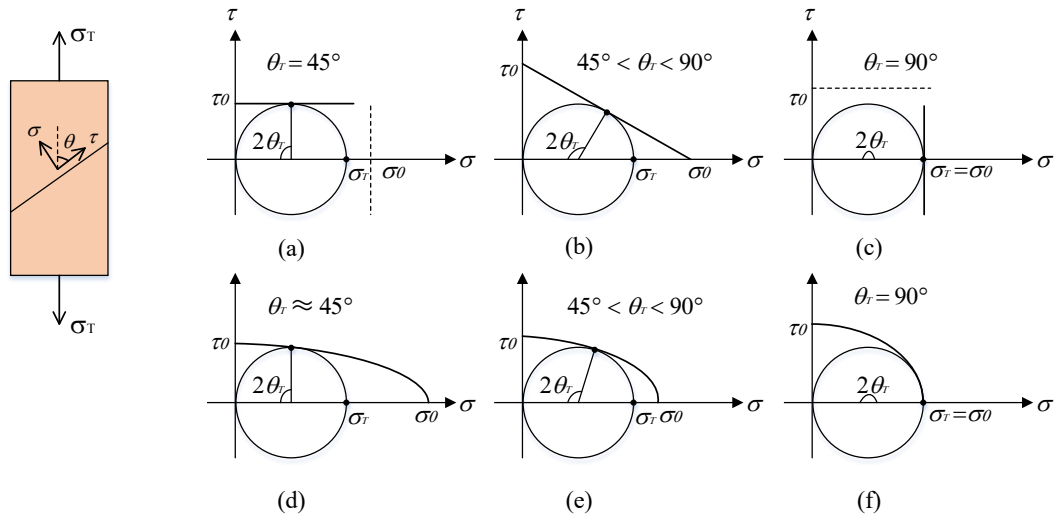


Figure 3.24 Schematic illustration of different tensile criteria: (a) Tresca criterion; (b) Mohr-Coulomb criterion; (c) maximum normal stress criterion; (d) ellipse criterion with $\alpha \rightarrow 0$; (e) ellipse criterion with $0 < \alpha < \sqrt{2}/2$; (f) ellipse criterion with $\alpha \geq \sqrt{2}/2$ (Zhang and Eckert, 2005).

Table 3.1 The different ratio a for the corresponding four classical criteria

Classical criteria	Expression	Fracture angle θ_T	Ratio α
Tresca criterion	$\tau_{\max} \geq \tau_0$	$\theta_T \approx 45^\circ$	$\alpha \rightarrow 0$

The Mohr-Coulomb criterion	$\tau + u \cdot \sigma \geq \tau_0$	$45^\circ < \theta_T < 90^\circ$	$0 < \alpha < \sqrt{2}/2$
The maximum normal stress criterion	$\sigma_{\max} \geq \sigma_0$	$\theta_T = 90^\circ$	$\alpha \geq \sqrt{2}/2$
The von Mises criterion	$\begin{cases} \sigma^2 + 3\tau^2 \geq 3Y^2 \\ \sigma_0 = \sqrt{3}\tau_0 = Y \end{cases}$	$\theta_T = 60^\circ$	$\alpha = \sqrt{3}/3$

It can be seen from Table 3.1, the ellipse criterion in Eq.(3.22) depends on the ratio a and can be classified into four cases, which approximatively represent different classical failure criteria. When $a \rightarrow 0$, the tensile shear fracture angle θ_T is close to 45° , the ellipse criterion agrees well with the Tresca criterion, as shown in Figure 3.24 (d). When $0 < \alpha < \sqrt{2}/2$, θ_T ranges from 45° to 90° , the ellipse criterion is consistent with the Mohr-Coulomb criterion, as shown in Figure 3.24 (e). Tensile failure of this ellipse criterion depends on both the normal stress σ and the shear stress τ , while the dependence of shear stress on normal stress is not linear, different from the linear Mohr-Coulomb criterion. As shown in Figure 3.24 (f), when $\alpha \geq \sqrt{2}/2$, θ_T equals 90° , the presented ellipse criterion is in good agreement with the maximum normal stress criterion. Meanwhile, when $\alpha = \sqrt{3}/3$, θ_T equals to 60° , which indicates that the von Mises criterion is a special case of the ellipse criterion. The ellipse criterion is a unified tensile fracture criterion describing the four failure modes mentioned above. Based on the penalty function method in the DDA, the Eq. (3.22) incorporated to determine the tensile failure in DDA can be expressed as follows:

$$\frac{\alpha^2 k_n^2 d_n^2}{c^2 l^2} + \frac{k_s^2 d_s^2}{c^2 l^2} \geq 1, (d_n < 0, c > 0) \quad (3.23)$$

where a is the ratio of the critical shear stress to the tensile fracture stress of the material, calculated by $a = \tau_0 / \sigma_0$ ($a > 0$) based on the experiment result of material characteristic. The strength parameters, such as cohesion c , friction angle ϕ and the

ratio a are used for representing the shear and tensile strength of materials in the calculation. It should be noted that the shear stress induced by friction on the contact interface will be regarded as zero when $d_n < 0$. The normal spring is subjected to tensile stress and the friction angle j is then assumed not to affect the shear stress τ_0 and $\tau_0 = cl$. When $c = 0$, the Mohr-Coulomb criterion and the maximum normal stress criterion in the original DDA will be used.

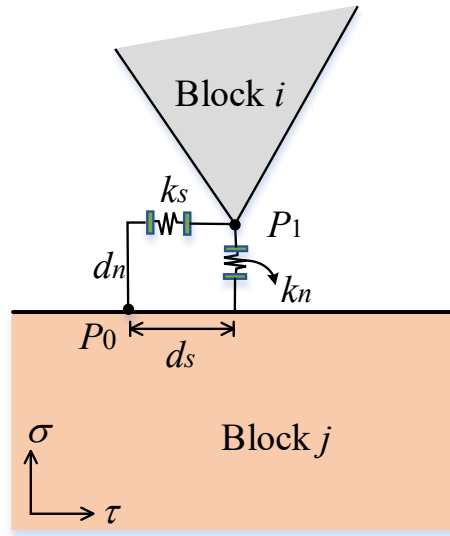


Figure 3.25 Determination of the tensile failure in I-DDA

In the I-DDA, the Mohr-Coulomb failure criterion remains unchanged for evaluating the shear failure as the original DDA method. When the normal penetration is positive, namely $d_n \geq 0$, the contact state is ‘closed’. The two blocks i and j interact through shear stress in the tangential direction and compressive stress in the normal direction. In this situation, the original Mohr-Coulomb failure criterion is further adopted to determine whether the contact state is ‘locked’ or ‘sliding’. As aforementioned, if the Eq. (3.3) is satisfied, the current contact state can be determined as ‘locked’; if the Eq. (3.4) is satisfied, it denotes ‘sliding’ contact state. However, when normal penetration is not positive ($d_n < 0$), as shown in Figure 3.25, the two blocks i and j interact with shear stress in the tangential direction and tensile stress in the normal direction. The new tensile fracture model, which unified four classical criteria, is adopted to replace the original maximum normal stress criterion. If the normal and shear spring force satisfy the new criterion

in Eq. (3.5), the contact state ‘open’ will be determined in the open-close iteration process; otherwise, the contact state remains the same. The I-DDA model with incorporation of the unified tensile fracture model can better describe the tensile fracture behaviors of various materials. The computational flow chart is presented in Figure 3.26.

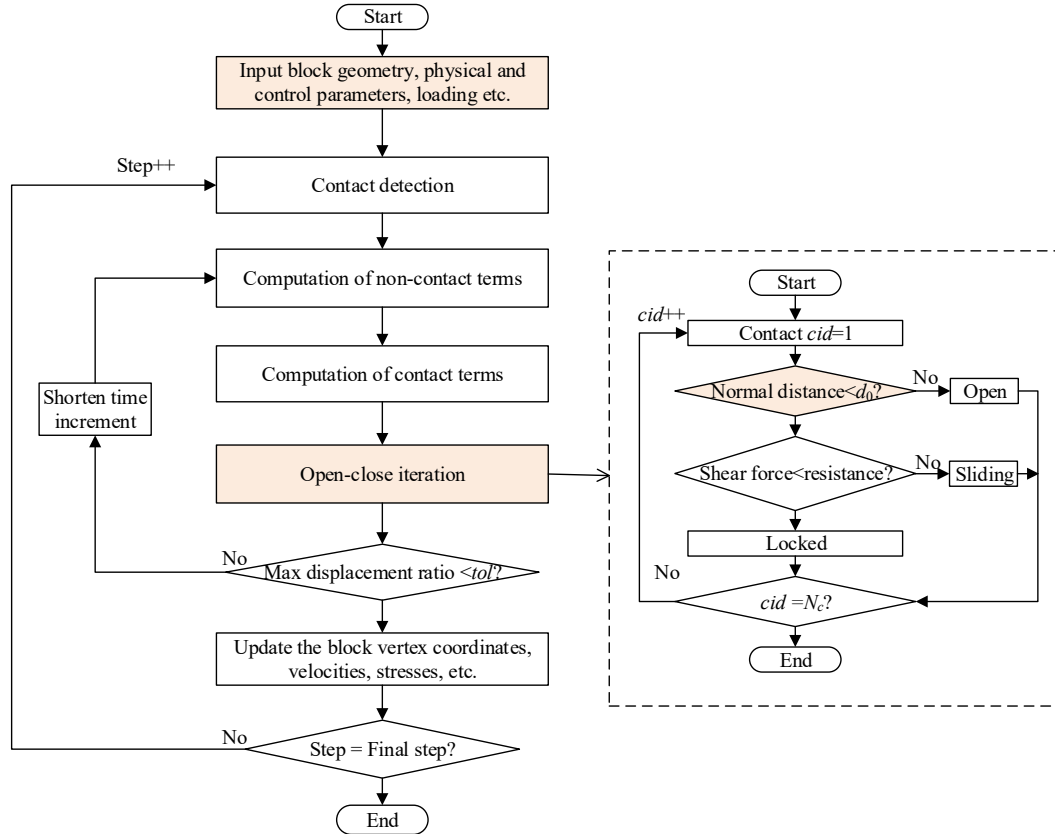


Figure 3.26 Flow chart of I-DDA incorporating the unified tensile fracture model. The processes in shaded boxes are modified

3.5.2. ARTIFICIAL MESHING FOR DDA SIMULATION

The original DDA method is capable of simulating rock fractures with pre-existing joints. For intact rock or concrete, the artificial joints are generally used for the DDA method to simulate the fracture behavior of the continuous materials. Generally, two types of joints form the edges of the triangular blocks, i.e., real and artificial joints. The pre-existing joints and model boundaries are defined as real joints, whereas the assistant joints and block boundaries separating the continuous

region are represented by artificial joints. The artificial joints provide paths for crack generation and propagation.

The method for generating the triangular blocks in this context is based on DISTMESH ([Persson, 2004](#)), a code for the generation and manipulation of unstructured triangular meshes. A variety of geometric shapes and desired mesh densities can be defined for the DDA blocks. By using this advancing mesh method, the DDA blocks can be easily subdivided into small triangular blocks with artificial joints.

3.5.3. CRACK INITIATION AND PROPAGATION

All the artificial joints are assigned with the same strength of the intact material to represent the continuous state across the joints. The crack generation is controlled by the failure of the artificial joint in DDA block system. Generally, the crack initiation procedure for numerical methods can be concluded as two cases ([Bao and Zhao, 2013](#)). One case in a continuum media is that cracks are not independent of each other. Once a crack occurs somewhere, it will cause the stress to redistribute in the area around it and will affect the behavior of the crack propagation in the nearby field. The other case is that many cracks can appear in a time step. Cracks will initiate when the joint contacts separate along the block according to the failure criteria.

In I-DDA method, the model for crack generation belongs to the second case, which is a multiple crack model. Once the stress states at these artificial joints reach the critical stress state of the unified tensile criterion or the Mohr-Coulomb shear failure criterion, the contacted joints will separate along block boundaries, thereby generating new cracks. At each time step, multiple cracks or no crack may be generated depending on the stress state.

3.5.4. NUMERICAL EXAMPLES

3.5.4.1. BLOCK SHEAR AND TENSILE TEST

As shown in Figure 3.27, a model consisting of two blocks is employed for simulating the numerical shear and tensile tests. Two linearly increasing forces P_1 , P_2 are acted simultaneously on the centroid of the upper block and $P_1 / P_2 = 1$. The bottom block is fixed by four fixed points. The two blocks have a density of 2500 kg/m^3 , Young's modulus of 5.0 GPa , Poisson's ratio of 0.25 . The contact interface between the two blocks has cohesion $c = 40 \text{ kPa}$ and friction angle $\varphi = 30^\circ$. In the calculation, the control parameters are: spring stiffness $k_n = 1.0 \text{ GPa}$, stiffness of the shear contact spring $k_\tau = 0.4k_n$, maximum time step size $\Delta t = 0.0001 \text{ s}$, and maximum allowed displacement ration of 0.0001 .

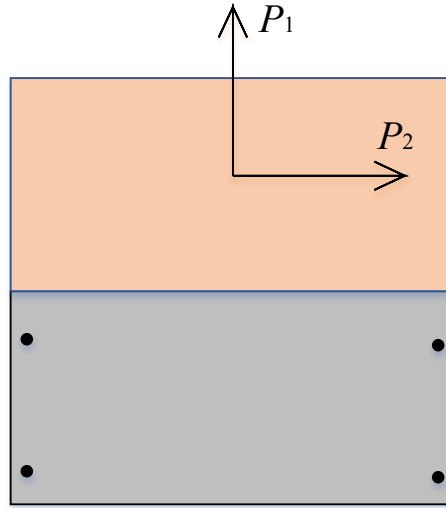


Figure 3.27 Model of shear and tensile tests

The upper block will be separated from the bottom block under the forces P_1 and P_2 . The tensile stress σ and shear stress τ at the moment of tensile failure for the material with different values of the ratio $\alpha = \tau_0 / \sigma_0$ (ranging from 0.01 to 10) are obtained by the I-DDA method. The different ratios α can be used to represent tensile properties of different materials: when $\alpha = 0.01 \sim 0.1$, the materials are regarded as shear controlled; when $\alpha = 0.1 \sim 1$, the materials can be identified as shear controlled and tension influenced; when $\alpha = 1 \sim 1000$, the materials can be considered as tension controlled. According to Eq. (3.22), the theoretical results of tensile stress σ and shear stress τ at the moment of tensile failure for the material with different ratios α can be calculated. At the critical state, the ratios $(\tau / \tau_0)^2$ and

$(\sigma/\sigma_0)^2$ can be derived as follows:

$$\begin{cases} \frac{\tau^2}{\tau_0^2} = \frac{1}{1+\alpha^2} \\ \frac{\sigma^2}{\sigma_0^2} = \frac{\alpha^2}{1+\alpha^2} \end{cases} \quad (3.24)$$

The simulated results are compared with the theoretical results to verify the accuracy and effectiveness of the I-DDA method. As shown in Figure 3.28, the simulated results are convergent to the theoretical solutions, which suggests that the accuracy of the I-DDA method is satisfied.

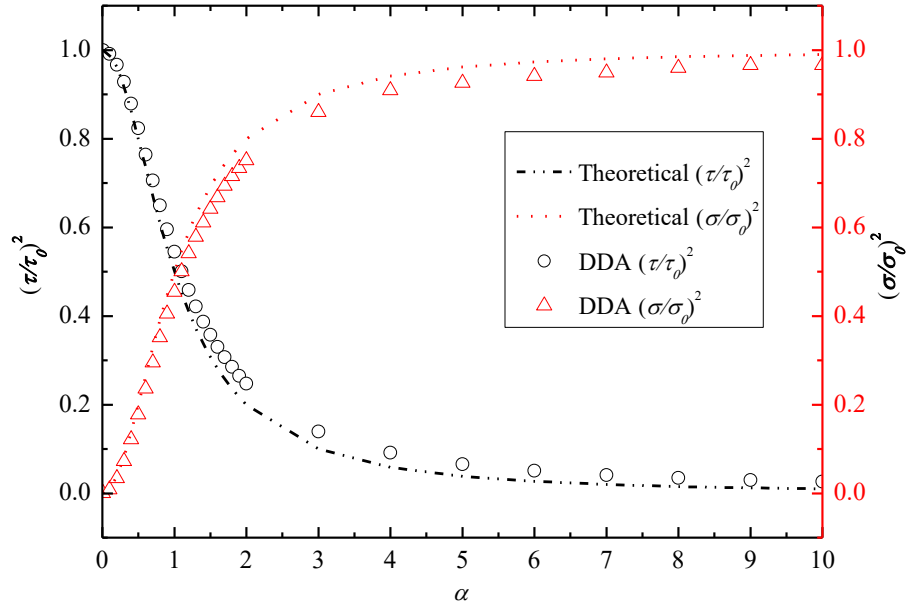


Figure 3.28 Result comparison of shear and tensile failure tests with different α

3.5.4.2. SIMULATION OF BRAZIL DISC SPLIT TEST USING I-DDA

The Brazil disc split test is a simple indirect testing method to determine the tensile strength of brittle materials such as concrete, rock, and rock-like materials. Based on previous studies on the Brazilian test, [Li \(2013\)](#) pointed out that the Brazilian test is a popular means to determine the tensile strength of rocks because of its easier specimen preparation and experimental procedure as compared with the direct uniaxial tensile test. Numerical approaches have been identified as one of the main research tools for the Brazil disc split tests. To investigate the applicability

of the I-DDA method in the modeling of the Brazil disc split tests, the failure processes of three types of rock discs under diametrical loadings are simulated, and the simulation results are discussed by comparing them with experimental and previous numerical results.

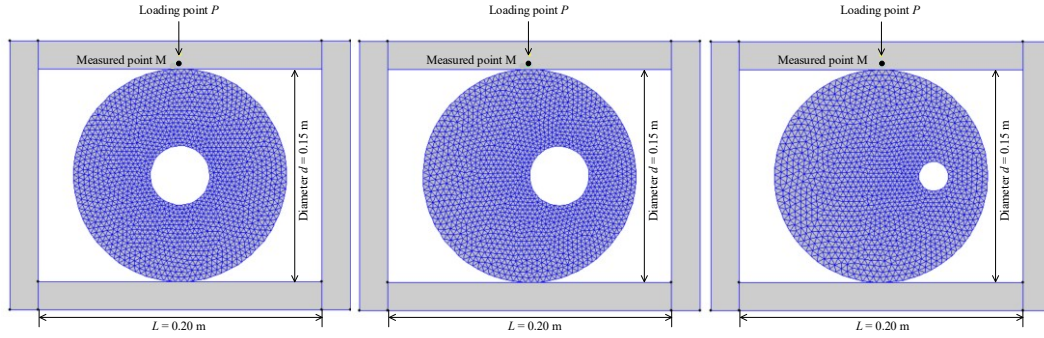


Figure 3.29 Three types of rock discs under diametrical loadings

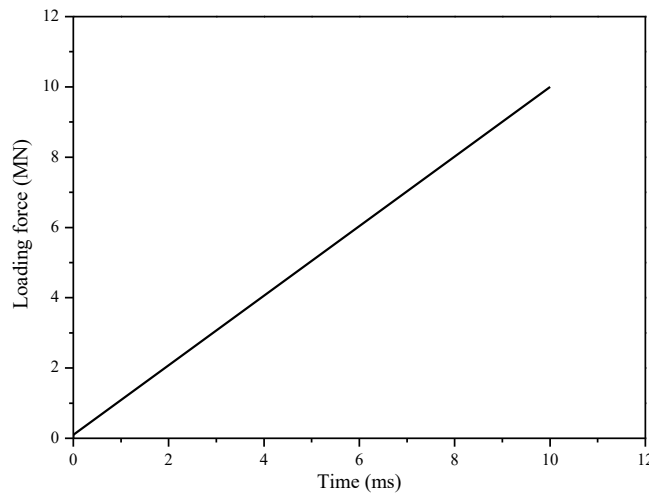


Figure 3.30 The increasing force P at the centroid of the loading plate

As shown in Figure 3.29, the discs are discretized into small triangular blocks randomly. As a result, the distribution of artificial joints that provide paths for crack generation and propagation in those models can be regarded as randomized. As shown in Figure 3.30, an increasing force P at the centroid of the loading plate is applied in the vertical direction. A measured point M at arbitrary point of the loading plate is also set to record the displacement and stress of the loading plate. When the disc is fractured, and a penetrative crack is observed, the peak loading force can be determined. Using the peak loading force, the simulated indirect tensile

strength of the material can be obtained by the following suggested formula for calculating the splitting tensile strength σ_t (MPa) based on the Brazilian test ([ASTM 2008](#)):

$$\sigma_t = \frac{P}{\pi R t} \quad (3.25)$$

where P is the load at failure, and in the I-DDA method $P = \sigma_n \times L$, σ_n is the peak value of measured normal stress of loading plate, and L is the edge length of the loading plate; R is the radius of the test specimen; t is the thickness of the test specimen and $t = 1$ m for the two-dimensional DDA.

As shown in Figure 3.29 (a), a Brazil disc has a diameter of 150 mm, in which a central hole with a diameter of 42 mm is drilled. The Brazil disc was discretized into 2858 triangular blocks. The disc is a homogeneous material with the density $\rho = 2500 \text{ kg/m}^3$, Young's modulus $E = 70 \text{ GPa}$, and the Poisson's ratio $\nu = 0.28$. The control parameters are: normal contact spring stiffness $k_n = 11 \text{ GPa}$, shear contact spring stiffness $k_t = 0.4 k_n$, maximum time step size $\Delta t = 0.0001 \text{ s}$, maximum allowed time-step displacement ratio of 0.001. Both cohesion and tensile strength of the interface of the Brazil disc are ignored. The material parameters of the loading plates are as follows: density $\rho = 2500 \text{ kg/m}^3$, Young's modulus $E = 100 \text{ GPa}$, and the Poisson's ratio $\nu = 0.2$. The strength parameters of artificial joints are adopted as follows: cohesion $c = 24 \text{ MPa}$, internal friction angle $\varphi = 30^\circ$, tensile strength $T = 15 \text{ MPa}$ (namely, the ratio $\alpha = \tau_0 / \sigma_0 = 1.6$). According to Table 3.1, the unified tensile criterion in I-DDA is approximately in agreement with the maximum normal stress criterion as $\alpha \geq \sqrt{2}/2$ in this simulation. The values of those strength parameters are taken from the related literature ([Jiao, 2012](#)), which were obtained from the experiment on rock material.

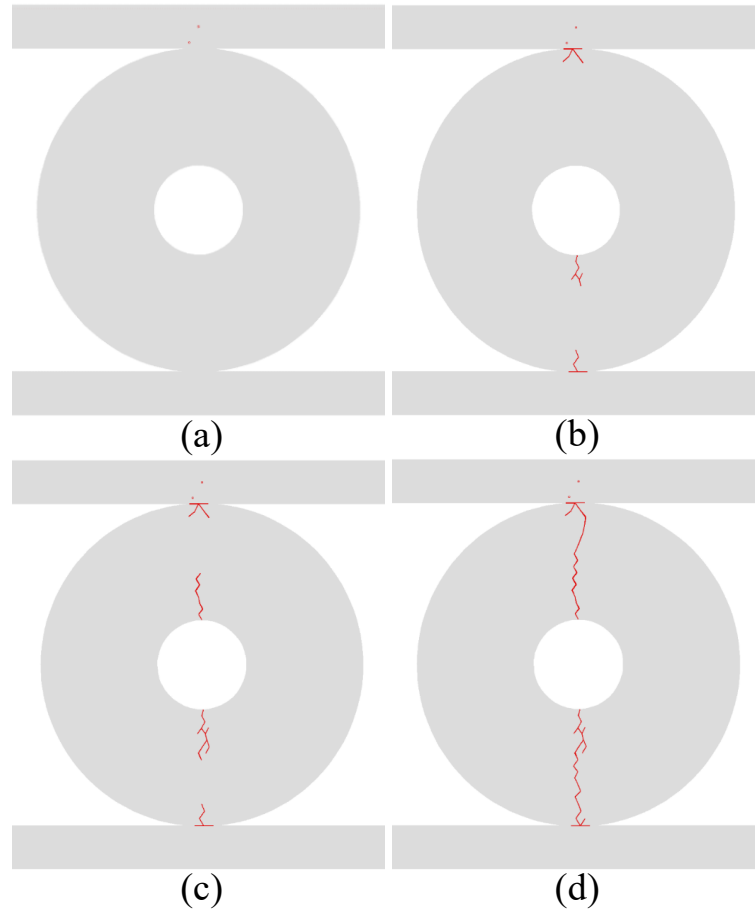


Figure 3.31 Simulated results of a diametrically loaded Brazilian disc with a central hole: (a) Intact disc; (b) Crack initiation; (c) Crack propagation; (d) Final cracks

Figure 3.31 shows the simulated process of cracks initiation and propagation path in a diametrically loaded Brazilian disc with a central hole. The cracks firstly initiate at the top and bottom boundaries of the central hole under a diametrical loading. The tensile stress that satisfies the tensile failure criterion starts at the center of the disc. As loading continually increases, the cracks propagate to loading points along the vertical direction. Finally, the disc is diametrically split into two parts, which is consistent with the phenomenon observed in physical tests and previous numerical analysis ([Jiao 2012](#)). The crack propagation path can be in any direction determined by the failure criterion as the sub-blocks are generated randomly. In the simulation, the crack propagation path is exclusively along the

vertical diameter of the disc, suggesting that the effect of the discretization approach on crack propagation is insignificant and negligible here.

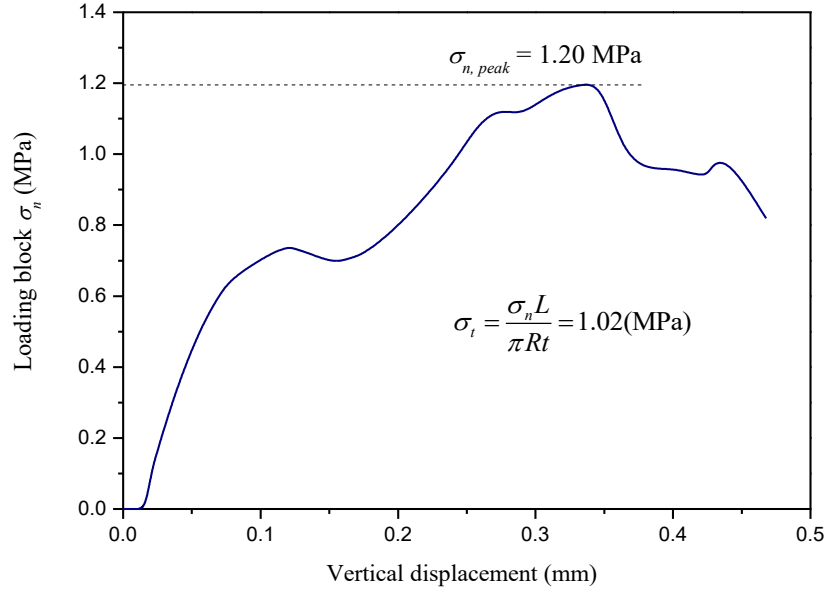


Figure 3.32 Relationship between stress and vertical displacement of the loading block

Figure 3.32 shows the relationship between the stress and vertical displacement of the loading block in the simulation of the Brazil disc with a central hole. It can be seen that as the displacement increases, the stress increases gradually from 0 MPa to a peak value of 1.20 MPa with a slight fluctuation, which is probably caused by the crack generation and propagation. Once the penetrative crack is formed, the stress decreases. With the peak stress determined, the peak loading force is obtained as $P = 240$ kN. Based on the suggested formula for calculating the splitting tensile strength, the simulated tensile strength of this disc model is 1.02 MPa. Cai (2013) pointed out that the simulated tensile strength of the Brazilian disc test result will be affected by the geometry and frictional resistance of the pre-existing crack when using the numerical method. It could be difficult to quantitatively analyze the effect of those two factors in this model as the discretization is randomized. However, the result of simulated tensile strength obtained from the I-DDA method could be a useful reference for evaluating the tensile strength of a particular disc.

The failure process of Brazil disc with a large eccentric hole is simulated using the I-DDA method in this section. The same experiments have been conducted by many researchers ([Van de steen et al. 2005](#); [Jiao 2012](#)) to investigate the effect of the eccentricity and the hole radius on the location of the failure, which proved that the disc has different fracture pattern with different eccentricity and diameter of the eccentric hole. To further verify the applicability of the I-DDA method, similar simulations are performed. As shown in Figure 3.29(b), the Brazil disc has a diameter of 150 mm. The diameter of the eccentric hole is 42 mm, with an eccentricity of 21 mm. The Brazil disc is randomly divided into 2789 triangular blocks. The material properties and strength parameters take the same values as those in the previous simulation.

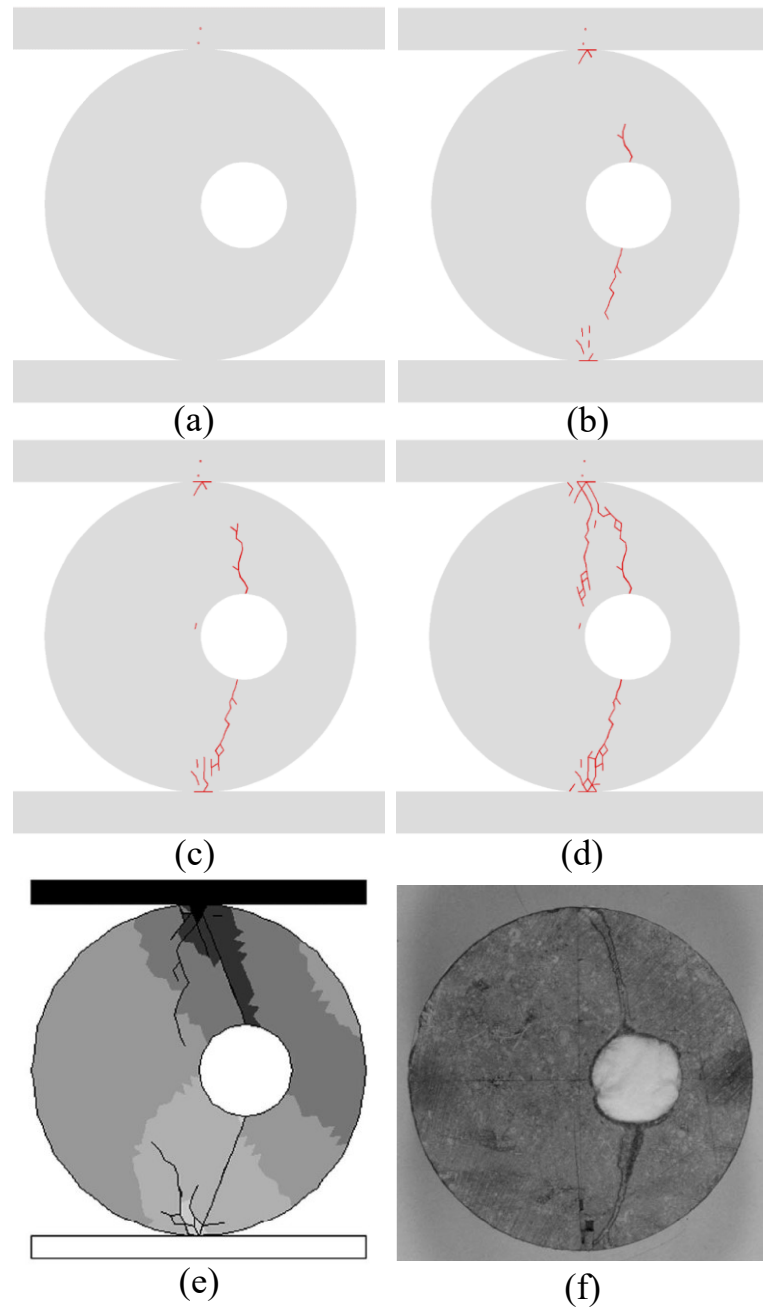


Figure 3.33 Simulated results of a diametrically loaded Brazilian disc with a large eccentric hole: (a) Intact disc; (b) Crack initiation; (c) Crack propagation; (d) Final cracks; (e) Numerical result ([Jiao, 2012](#)); (f) Experimental result ([Van de Steen, 2005](#))

The fracture patterns at different stages during the simulation are obtained, illustrated in Figure 3.33. The crack initiates from the top and bottom sides of the hole and propagates to the loading points at the top and the bottom of the disc. It

can be theoretically explained that because the hole of this disc is relatively large, the maximum tensile stress concentrates near the top and bottom sides of the hole. Thus, the cracks are initiated there and propagate to the loading points. Secondly, another secondary crack extends from the top-loading point of the disc to approximately the center of the sample, which is also observed in other numerical and experimental researches, as shown in Figure 3.32 (e)-(f). It can be seen that the tensile stress also concentrates along the vertical diameter in this configuration. When the maximum tensile stress concentrations on a loaded diameter are larger than those tensile stress concentrations on the hole, the cracks will occur along the loaded diameter. In this simulation, the cracks propagation is not consistent with vertical direction because of the effect of large eccentric hole. The stress field has been significantly affected by the large eccentric hole. As there is a large eccentric hole in the Brazil disc, the maximum tensile stress mainly concentrations on top and bottom sides of the eccentric hole and the cracks initiate from the top and bottom sides of the eccentric hole and propagate to the loading points in the simulated results, which is in well agreement with the experimental result (Van de Steen, 2005).

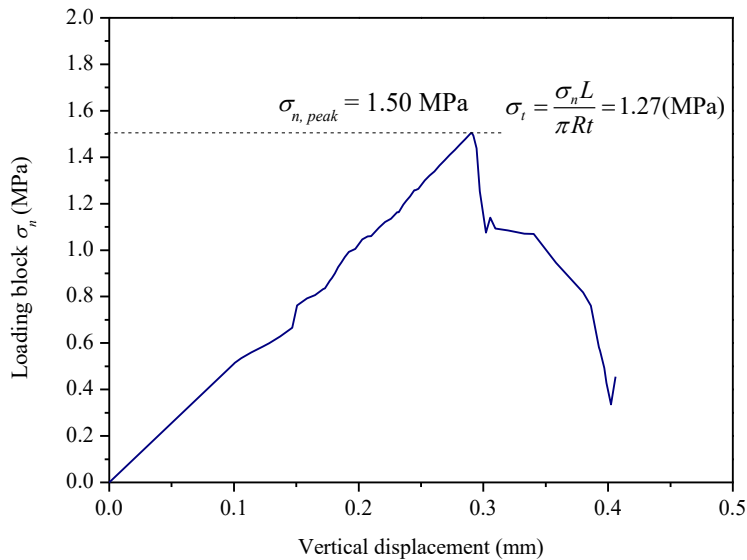


Figure 3.34 Relationship between stress and vertical displacement of the loading block

Figure 3.34 shows the relationship between the stress and vertical displacement of the loading block in the simulation of the Brazil disc with a large

eccentric hole. As the displacement increases, the stress increases gradually from 0 MPa to a peak value of 1.50 MPa. Once the penetrative cracks are formed from the top and bottom sides of the eccentric hole to the loading points of the top and the bottom disc, respectively, the stress begins to decrease. If the loading is continuously increased, the split blocks will break into some smaller parts. During this evolution process, the stress will fluctuate in small stress range and small stress increases may be observed near the end of the test because of the occurrence of the secondary cracks. However, the stress fluctuation at the end of the test will not influence the determination of peak stress as the increase is very small. With the peak stress determined, the peak loading force is obtained as $P = 300$ kN. Based on the suggested formula for calculating the splitting tensile strength, the simulated tensile strength of this disc model is 1.27 MPa.

In addition, the Brazil disc with a small eccentric hole is also reproduced by the I-DDA method. As shown in Figure 3.29 (c), the Brazil disc has a diameter of 150 mm; the diameter of the eccentric hole is 21 mm, with an eccentricity of 21 mm. The material properties and strength parameters take the same values as those in the previous simulation.

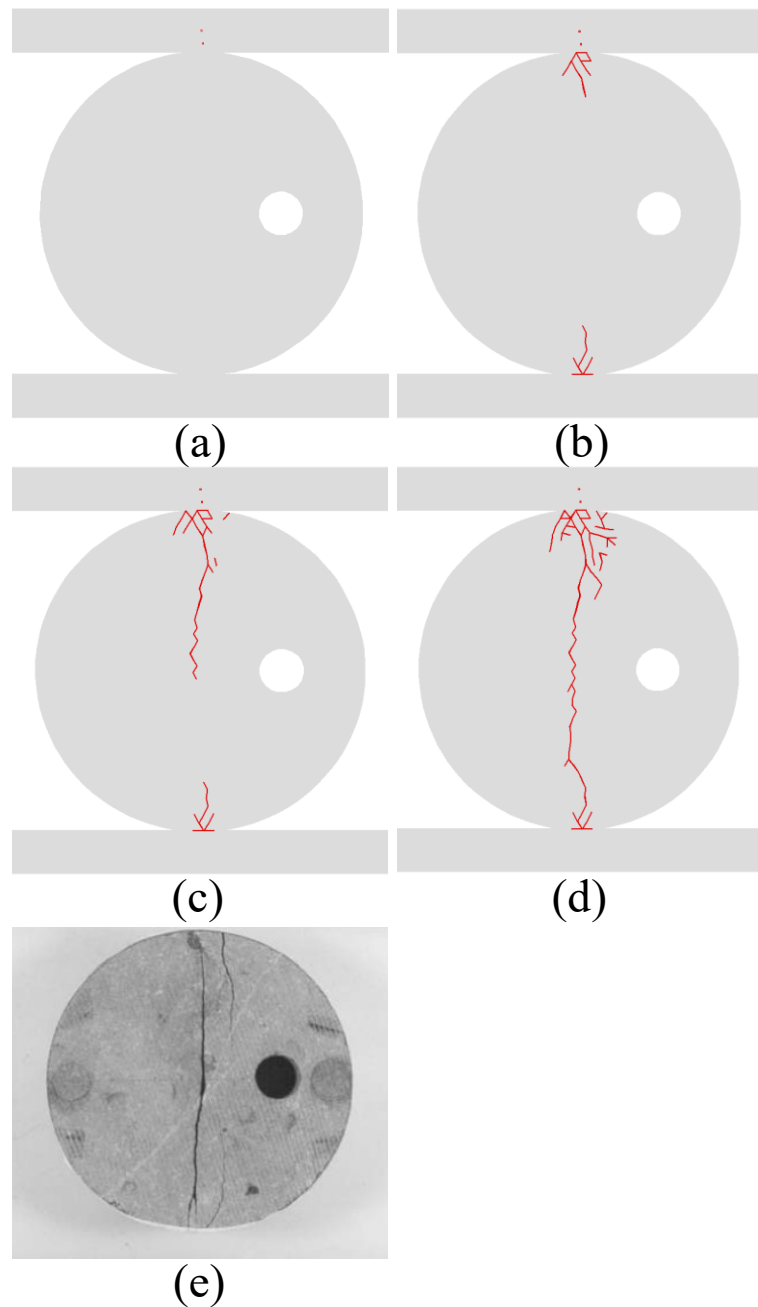


Figure 3.35 Simulated results of a diametrically loaded Brazilian disc with a small eccentric hole: (a) Intact disc; (b) Crack initiation; (c) Crack propagation; (d) Final cracks; (e) Experimental result ([Van de Steen, 2005](#))

As shown in Figure 3.35, the simulated results agree well with experimental results from Van Fe Steen (2005). If the hole radius is small enough, the cracks initiate and propagate along the vertical loaded diameter. The cracks first initiate at the top and bottom of the loaded disc and propagate continuously from the top and

bottom fracture points to the center of the disc under the diametrical loading. The disc is finally split into two parts when the cracks coalesce to form a diametrically penetrative crack observed in Figure 3.35 (d), which is consistent with the experiment results in Figure 3.35 (e). It can be concluded that the influence of the small eccentric hole on crack propagation is negligible compared with the eccentric hole in Figure 3.29 (b).

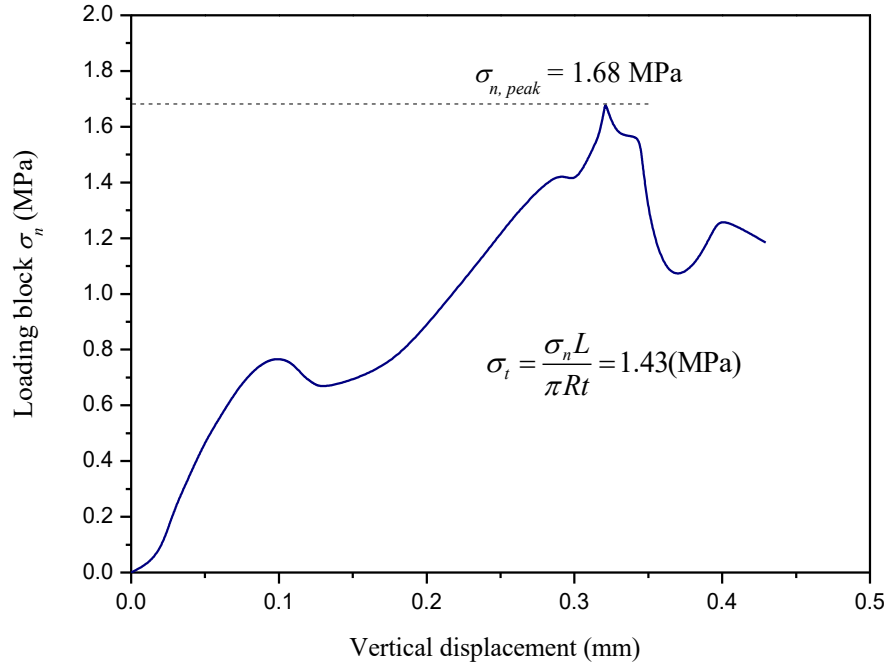


Figure 3.36 Relationship between stress and vertical displacement of the loading block

Figure 3.36 shows the relationship between the stress and vertical displacement of the loading block in the simulation of the Brazil disc with a small eccentric hole. A similar tendency with the previous configurations is observed. As the displacement increases, the stress increases gradually from 0 MPa to a peak value of 1.68 MPa. Once the penetrative cracks are formed, the stress begins to decrease. With the peak stress determined, the peak loading force is obtained $P = 300$ kN. Based on the suggested formula for calculating the splitting tensile strength, the simulated tensile strength of this disc model is calculated as 1.27 MPa. The simulated tensile strengths of the three Brazil discs are different as the geometry of the Brazil disc is one of the main factors affecting the tensile strength of a disc.

For the three Brazil disc split tests, it can be observed that the detailed processes of rock fracture, such as crack initiation, propagation and coalescence, can be simulated effectively through the I-DDA method. Meanwhile, the simulated indirect tensile strength of the material can also be obtained. The I-DDA also shows a significant advantage in simulating the rock fracture problems of various materials as the unified tensile fracture criterion is incorporated. It is an alternative quantitative analysis approach for rock fragmentation problems.

3.5.4.3. RESULT COMPARISON WITH O-DDA

Compared with the standard maximum tensile criterion in the original DDA (O-DDA), the unified fracture criterion in I-DDA is used to describe the tensile failure behavior. For demonstrating the efficiency and accuracy of the I-DDA, a comparative analysis on simulation of Brazil disc split tests with I-DDA and O-DDA is also presented. In the view of most studies, the splitting fracture of rocks in the Brazil disc test is caused by the maximum tensile stress along the diametral loading line, thus the maximum tensile criterion is regarded as one of the appropriate criteria for describing the crack propagation in a Brazil disc ([Li and Wong, 2013](#)). In O-DDA, the reasonable results can be produced if the parameters are appropriate. In comparison, the unified fracture criterion unifies the four classical criteria, including the maximum tensile criterion. In this case, the ratio of tensile strength and shear strength is 1.6, indicating that the new unified fracture criterion approximately agrees with the maximum normal stress criterion according to Table 3.1. In theory, the unified fracture criterion in I-DDA is also available to describe the crack propagation of the Brazilian disc in this case. The material parameters and control parameters for O-DDA are the same with those in I-DDA.

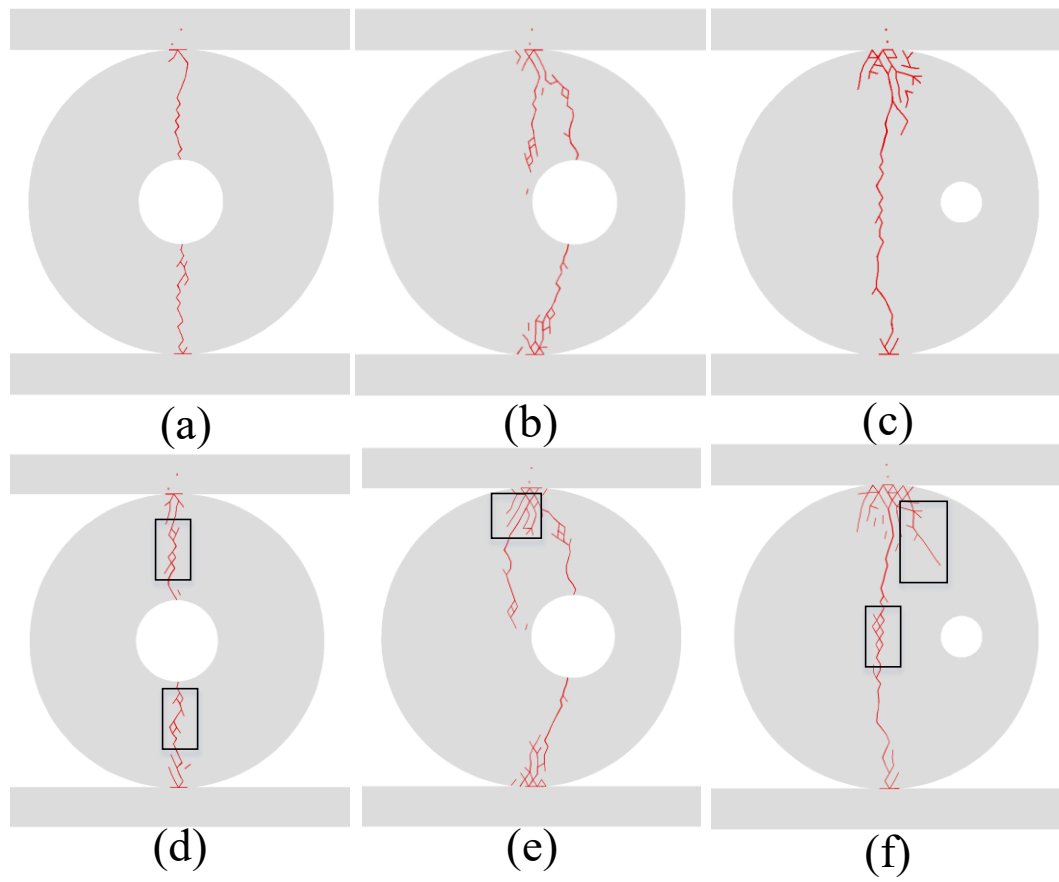


Figure 3.37 Comparison of crack patterns between I-DDA and O-DDA: (a) Model *a* with I-DDA; (b) Model *b* with I-DDA; (c) Model *c* with I-DDA; (d) Model *a* with O-DDA; (e) Model *b* with O-DDA; (f) Model *c* with O-DDA.

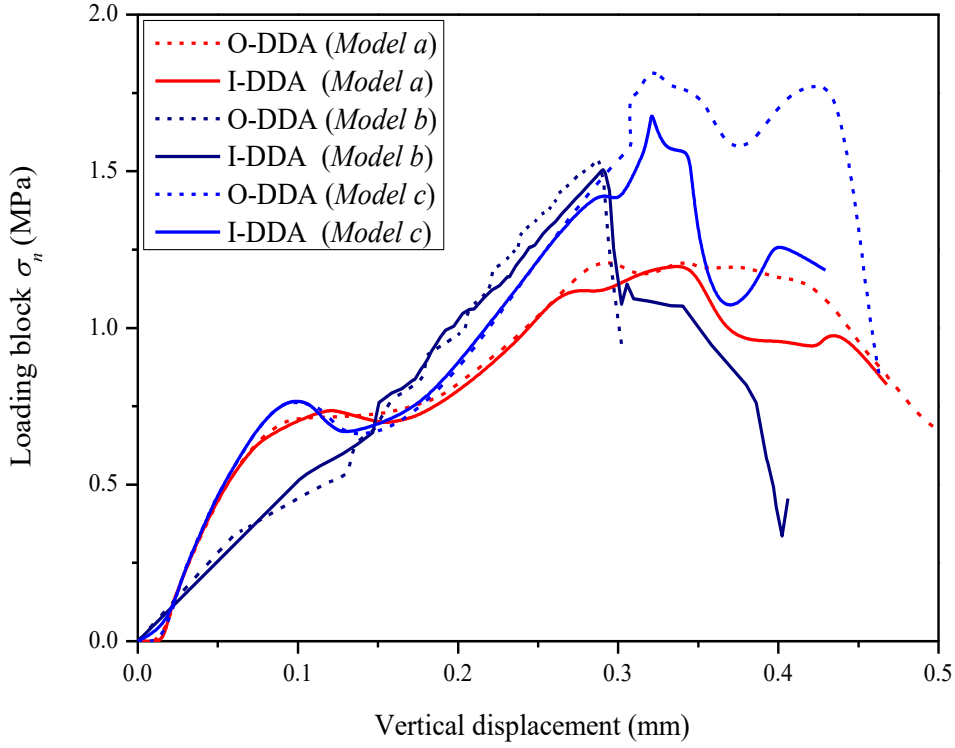


Figure 3.38 The comparison of normal stress and vertical displacement curves of the loading block between I-DDA and O-DDA

Figure 3.37 shows the simulated results of crack patterns in Brazil disc split tests at the final step when a diametrically penetrative crack occurs using I-DDA and O-DDA. It can be seen that the main crack patterns of I-DDA are consistent with the results of O-DDA. However, more secondary cracks around the main penetrative crack are observed in the results of the O-DDA method, which are presented in the rectangular region, as shown in Figure 3.38 (d), (e) and (f). Meanwhile, it can be seen from Figure 3.38 that the normal stress and vertical displacement curves of the loading block fluctuate significantly in O-DDA as the secondary cracks occur, especially for *Model a* and *Model c*. Compared with the results of O-DDA, the crack initiates and propagates linearly and fewer secondary cracks are observed in the results of I-DDA. This indicates that compared with the maximum normal stress criterion, the unified fracture criterion has better convergence and stability during the calculation.

3.5.4.4. PARAMETER ANALYSIS

The spring stiffness is an important parameter for DDA simulation. It is essential to select proper parameters for the stiffness of the contact spring. Many studies have been conducted to investigate the contact spring stiffness in DDA. Shi (1989) suggested that the very stiff contact spring must be larger than the Young's Modulus of the block to guarantee the very limited penetration between blocks. In contrast, Wu et al. (2004) proposed the soft contact spring to converge the unwanted contact forces quickly in DDA. Also, Akao et al. (2007) showed that soft contact spring resulted in appropriate dynamic simulation in DDA. Wu et al. (2013) investigated contact spring stiffness with different ratios of normal contact spring stiffness, k_n , to the Young's Modulus of the block, E . When k_n/E is smaller than 1.0, the unwanted stress of block in DDA converged quickly and the soft contact spring was proved to work well in DDA. While if the spring stiffness is too small, the block penetration occurs. Thus, the soft spring stiffness within an appropriate range is suggested in DDA simulation. To investigate the effect of spring stiffness on simulation of crack patterns in Brazil disc split tests, several simulations with different values of normal spring stiffness and Young's Modulus ($k_n/E = 0.05 \sim 0.5$) are performed based on the model in Figure 3.29 (a). Firstly, Young's Modulus of the blocks remains unchanged as $E = 70$ GPa. While the normal spring stiffness takes: (a) $k_n = 35$ GPa, $k_n/E = 0.5$; (b) $k_n = 7$ GPa, $k_n/E = 0.1$; (c) $k_n = 3.5$ GPa, $k_n/E = 0.05$. On the other hand, the spring stiffness remains unchanged as $k_n = 11$ GPa. The Young's Modulus of the blocks is considered as: (a) $E = 70$ GPa; (b) $E = 50$ GPa; (c) $E = 30$ GPa.

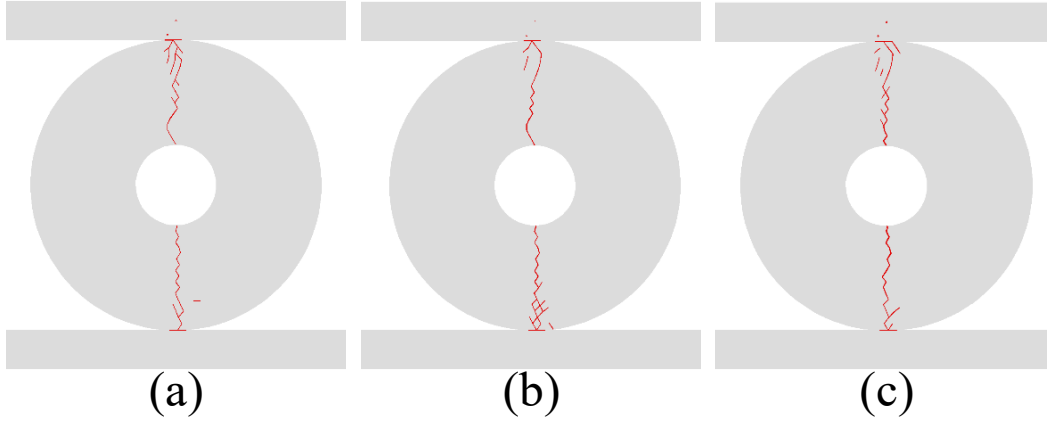


Figure 3.39 The results of I-DDA with different spring stiffness: (a) $k_n = 35$ GPa, $k_n/E = 0.5$; (b) $k_n = 7$ GPa, $k_n/E = 0.1$; (c) $k_n = 3.5$ GPa, $k_n/E = 0.05$.

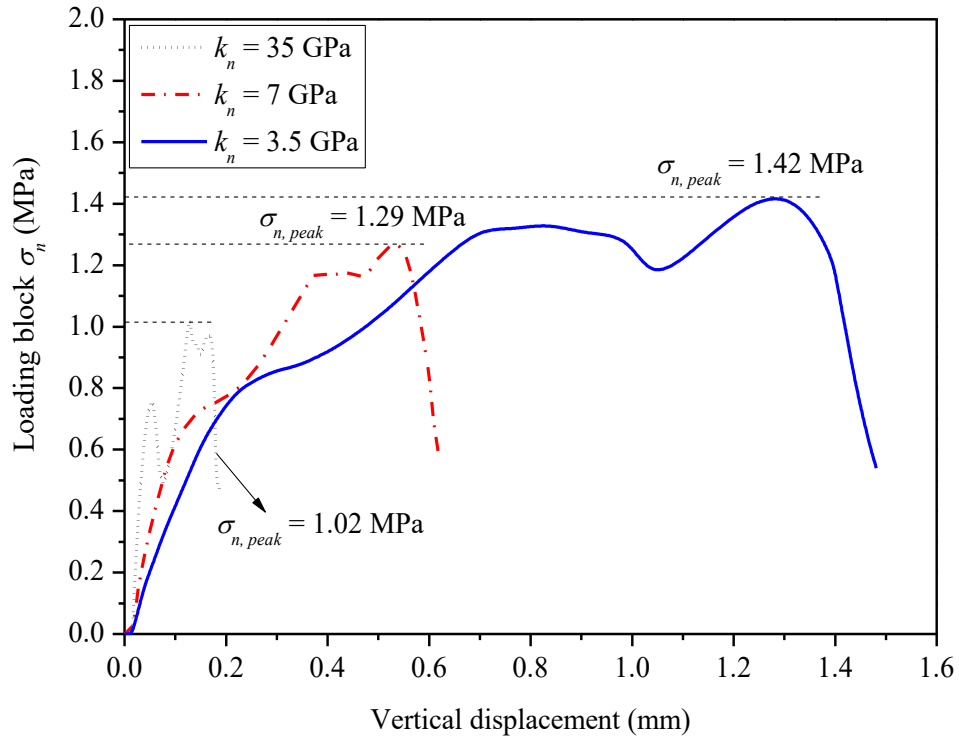


Figure 3.40 Relationship between normal stress and vertical displacement of the loading block with different spring stiffness: (a) $k_n = 35$ GPa, $k_n/E = 0.5$; (b) $k_n = 7$ GPa, $k_n/E = 0.1$; (c) $k_n = 3.5$ GPa, $k_n/E = 0.05$.

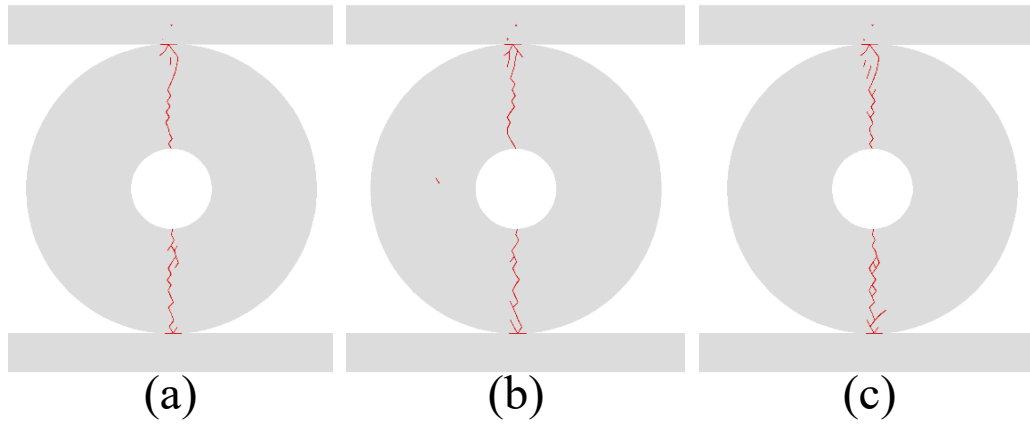


Figure 3.41 The results of I-DDA with different Young's Modulus: (a) $E = 70$ GPa; (b) $E = 50$ GPa; (c) $E = 30$ GPa.

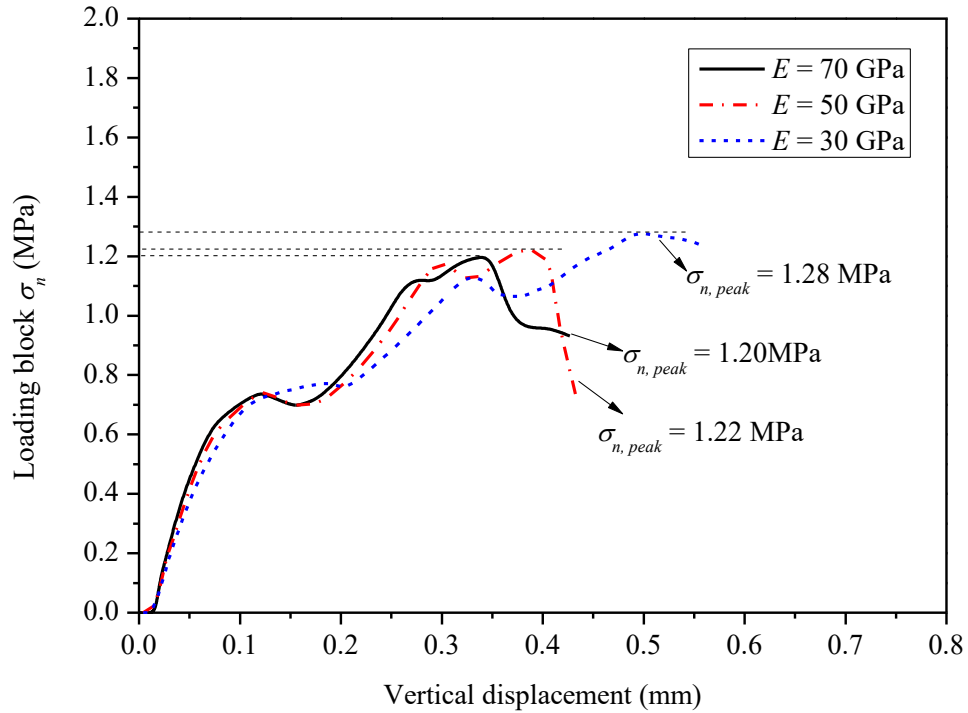


Figure 3.42 Relationship between normal stress and vertical displacement of the loading block with different Young's Modulus: (a) $E = 70$ GPa; (b) $E = 50$ GPa; (c) $E = 30$ GPa.

As shown in Figure 3.39, the crack patterns are consistent with the previous results in Figure 3.31 with a change of normal spring stiffness in an appropriate range. However, the change of normal spring stiffness has a significant effect on the maximal normal stress of the Brazil disc split test, as shown in Figure 3.40. As the normal spring stiffness decreases from 35 GPa to 3.5 GPa, the measured

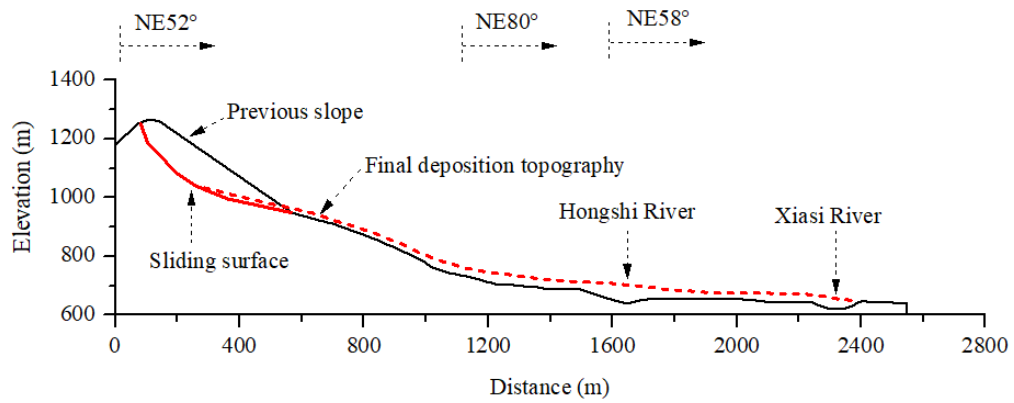
maximal normal stress of the Brazil disc increases from 1.02 MPa to 1.42 MPa. Different values of normal spring stiffness will produce different simulated tensile strength of the Brazil disc. Hence, the determination of normal spring stiffness should consider the material's tensile characteristics observed from the experiment in the simulation of the Brazil disc split test using I-DDA. In contrast, the change of Young's modulus has slight effect on both crack patterns and the maximal normal stress of the Brazil disc split test, as shown in Figure 3.41 and Figure 3.42. The crack patterns of I-DDA with different Young's Modulus are in good agreement and the maximal normal stress slightly fluctuates from 1.20 MPa to 1.28 MPa. It can be concluded that the change of Young's modulus and the spring stiffness within an appropriate range is available for simulating the crack propagation in the Brazilian disc by using the proposed I-DDA. The change of Young's modulus has no significant influence on results, while the normal spring stiffness should be carefully selected based on the tensile strength from the experiment.

3.6. APPLICATION TO LANDSLIDE ANALYSIS

Earthquake-induced landslide is one of the catastrophic secondary hazards of an earthquake. The mechanism of landslide initiation during an earthquake has not been clearly clarified up to now. The main methods for earthquake-induced landslide analysis such as pseudo-static analysis and Newmark's method, focus on the effects of earthquake accelerations on the landslide mass. One limitation of these methods is their use of the Mohr-Coulomb failure criterion, which only accounts for shear failure, but the role of tensile failure is not considered ([Gipprich et al., 2008](#)). The DDA method considers both shear and tensile failures of landslide mass, but the maximum normal stress criterion in O-DDA is only suitable to pre-existing weak joints in rock.



(a)



(b)

Figure 3.43 (a) Air photo of Donghekou landslide (modified from Zhang et al., 2014 and Yin et al., 2009) (b) cross section (modified from Zhang et al., 2014 and Yin et al., 2009)

In contrast, the unified tensile fracture criterion in I-DDA can describe tensile behaviors of the rock mass, which has a great advantage in analyzing slope instability considering the tensile failure. To verify the validity of I-DDA in the practical application to a real landslide, the I-DDA is applied to Donghekou landslide analysis considering both the shear and tensile properties of landslide mass. As shown in Figure 3.43, the Donghekou landslide is a large-scale earthquake-induced landslide in the 2008 Wenchuan earthquake, which caused

about 780 deaths and catastrophic damage to the local villages. It is a typical tension-shearing slip landslide with long run-out and high velocity (Xu et al., 2009). As shown in Figure 3.43 (b), the rockslide has a height of 700 m between the toe and the main scarp, a sliding distance of 2400 m and a volume of 10 million m³ (Yin et al., 2009; Zhang et al., 2014). Based on the typical cross-section presented in Figure 3.43 (b), a DDA model, including a base block and sliding mass, is built as shown in Figure 3.44 (modified after Zhang et al., 2014). The sliding mass is discretized into multiple smaller discrete deformable blocks according to the Voronoi method. The nearest recorded ground motions in the horizontal and vertical directions from the MZQP station, as shown in Figure 3.45, are applied to the base block. From the previous simulation on Donghekou landslides, the block located at the toe of the sliding mass has the largest run-out (Peng et al., 2020). A measured point M1 is set on the block at the toe of the sliding mass to record the landslide run-out under earthquake.

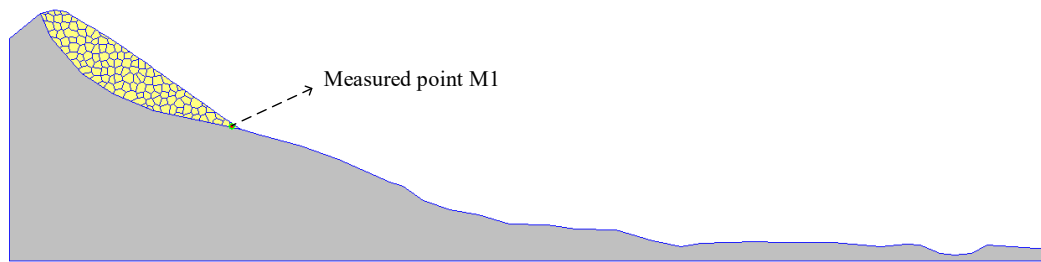


Figure 3.44 A simple numerical model of Donghekou landslide

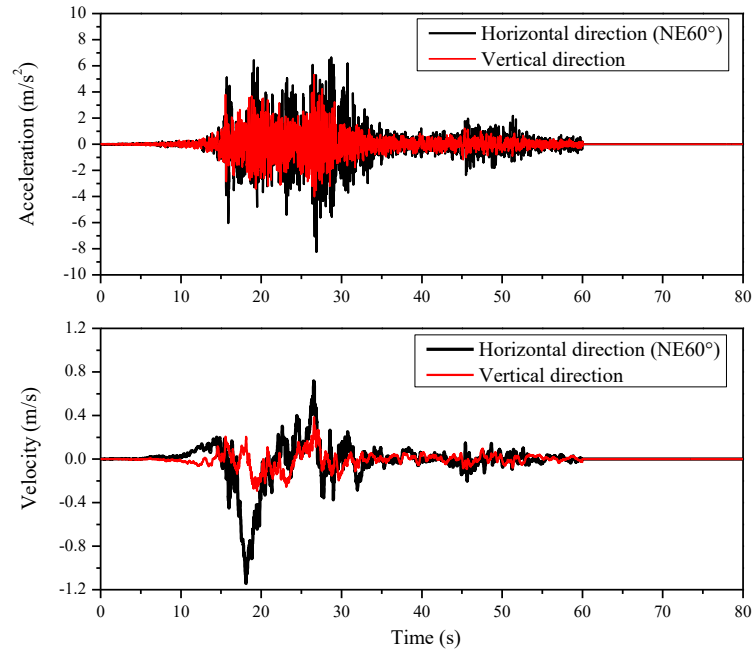


Figure 3.45 Ground motions in the horizontal and vertical direction from the MZQP station

The relevant calculation control parameters are: normal contact spring stiffness $k_n = 1.0$ GPa, maximum time step size $\Delta t = 0.01$ s, maximum allowed time-step displacement ratio of 0.001, and physical simulation time of 120.0 s. The mechanical properties of the sliding mass are as follows: density $\rho = 2150$ kg/m³, Young's modulus $E = 5.4$ GPa, Poisson's ratio $\nu = 0.28$, cohesive strength $c = 2.6$ MPa and friction angle $\phi = 12^\circ$. Those parameters are adopted from the previous literature (Zhang et al., 2014). How to determine the strength of the material is an important and primary question in the sliding analysis. The sliding mass of Donghekou rockslide-debris flow is mainly composed of phyllite and carbon slate. By using the shear test of the joint sample with different saturation from saturated to dried, the cohesive strength of phyllite is $c = 0.83$ -1.65 MPa and the cohesive strength of carbon slate $c = 2.09$ - 4.89 MPa (Sun et al., 2011). By back-calculating the shear strength c and friction angle ϕ based on the stability and the run-out scenario derived by DDA, an equivalent cohesive strength $c = 2.6$ MPa and friction angle $\phi = 12^\circ$ were estimated, but the tensile strength T was set as 0 in their studies without consideration of tensile strength. The ratio of shear stress and tensile stress

obtained from the experiment (Sun et al., 2011) is taken as $\alpha = 0.81$ ($T = 3.2$ MPa) is used in I-DDA.

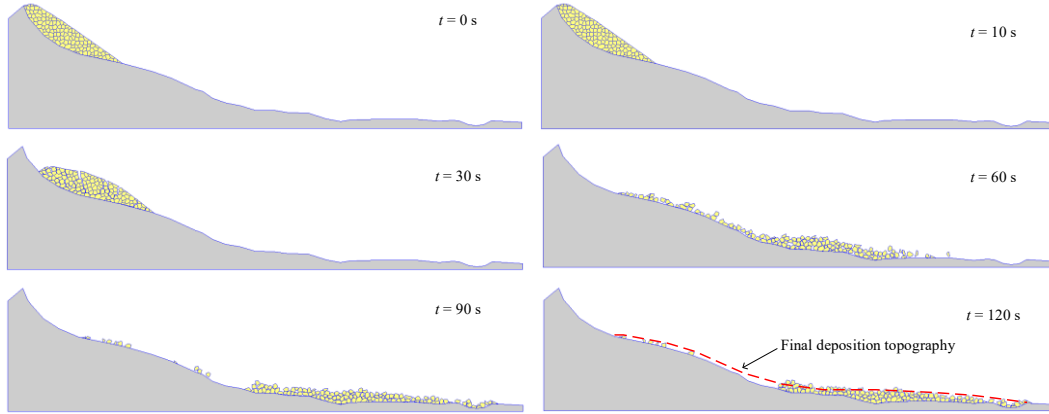


Figure 3.46 Simulated failure process of the Donghekou landslide

The failure process of the Donghekou landslide is simulated by the I-DDA with consideration of tensile and shear characteristics simultaneously, and the simulation sequences are displayed in Figure 3.46. The slope is stable before the earthquake and becomes unstable during the earthquake. The final run-out is approximately 2400 m. After overlapping the final deposition of simulation results with the topographic cross-section at the Donghekou landslide, the final deposition topography is in accordance with the actual topography after the landslide considering the tensile strength, which suggests that the I-DDA is available to conduct the landslide post-failure analysis considering the tensile and shear characteristic simultaneously.

To further verify the effect of tensile strength on slope stability, the Donghekou model with a wide range of tensile strength: (1) the ratio $\alpha = 1000$ ($T = 2.6$ kPa); (2) the ratio $\alpha = 1$ ($T = 2.6$ MPa); (3) the ratio $\alpha = 0.7$ ($T = 3.7$ MPa); (4) the ratio $\alpha = 0.3$ ($T = 8.7$ MPa), are also calculated in this study. For comparison, the Donghekou landslide is calculated by I-DDA and O-DDA.

Figure 3.47 shows the results of landslide simulation with a wide range of tensile strength from the I-DDA and O-DDA. The landslide is triggered at the time when the earthquake starts in all cases from the results of O-DDA. No significant difference is observed in the results of O-DDA as the tensile strength of slope

increases. While in O-DDA, the landslide is triggered at a different instant or remains stable with different tensile strengths. Compared with O-DDA, the I-DDA with the unified tensile criterion is more reasonable in theory for tensile failure analysis and the simulated results also indicate that the I-DDA is more available to simulate the tensile behavior in slope stability analysis.

As shown in Figure 3.47 (a), the tensile strength of the slope is 2.6 kPa, which is very small compared with the shear strength. The material is assumed to be tensile controlled, and the unified tensile criterion in I-DDA is consistent with the maximum normal stress criterion. The simulated results show that a tensile crack normal to the sliding surface is generated in the sliding mass at first under earthquake, and the slope then starts to collapse and evolves into a landslide. When the tensile strength of the slope is 2.6 MPa, the unified tensile criterion in I-DDA agrees with the maximum normal stress criterion. It can be seen from Figure 3.47 (c) that the rock mass starts to slide as the earthquake starts, and two tensile cracks are observed. The landslide initiation failure behaves as the main shear failure, while the tensile failure is also observed simultaneously. When the tensile strength is 3.7 MPa as shown in Figure 3.47 (e), the unified tensile criterion in I-DDA is consistent with the Mohr-Coulomb criterion. The slope starts to collapse with similar behavior, shear failure controlled with tensile cracks observed, but the initiation time is 20 seconds later. The slope has better stability with the increase of tensile strength. When the tensile strength is 8.7 MPa as shown in Figure 3.47 (g), the slope remains stable under the earthquake ground motion. It can be seen from Figure 3.47 that the tensile strength has a significant influence on the slope stability analysis. With different tensile strength, the failure process also has different behavior.

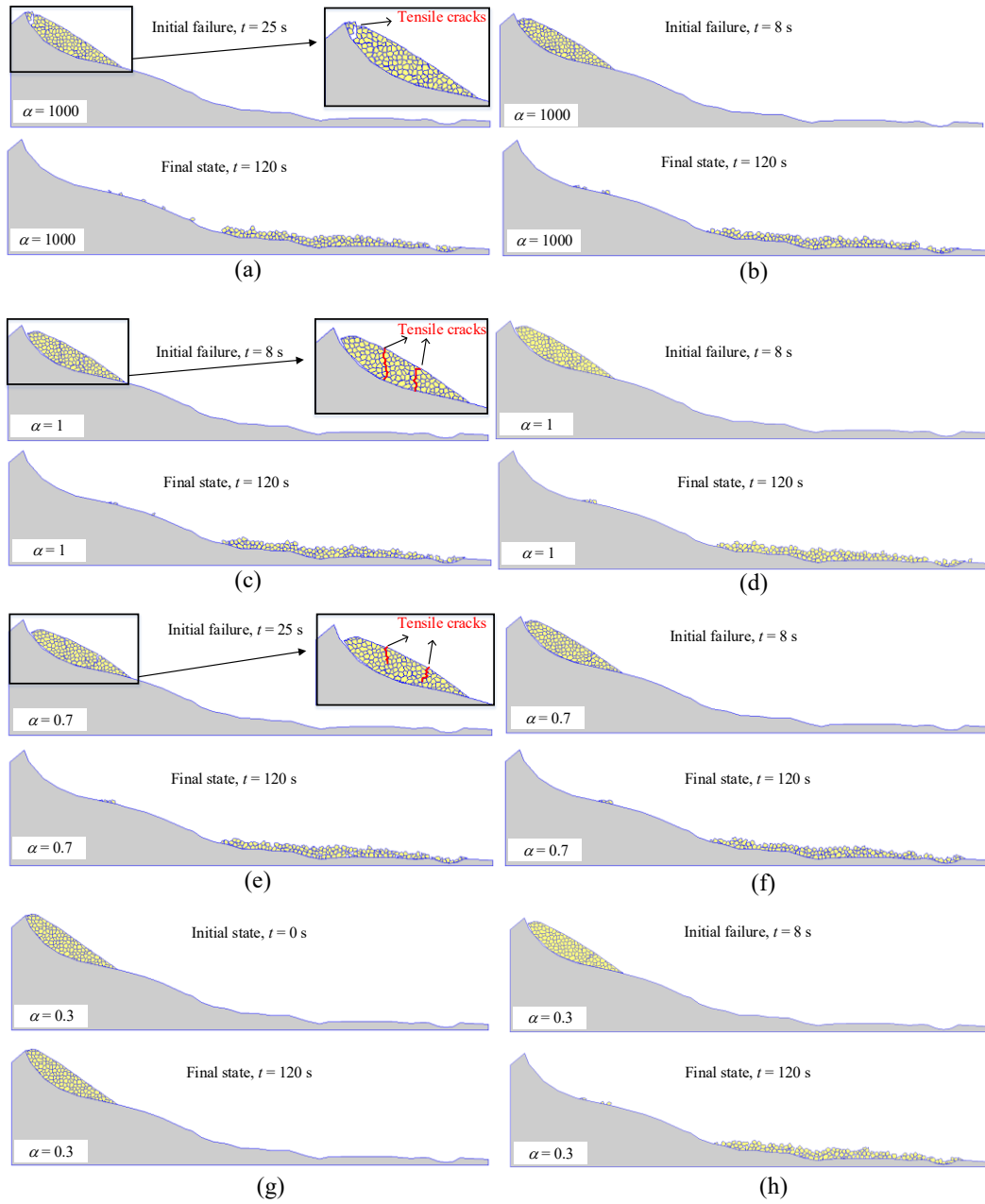


Figure 3.47 Results of landslide simulation with a wide range of tensile strength: (a) the ratio $\alpha = 1000$ ($T = 2.6$ kPa) with I-DDA; (b) the ratio $\alpha = 1000$ ($T = 2.6$ kPa) with O-DDA; (c) the ratio $\alpha = 1$ ($T = 2.6$ MPa) with I-DDA; (d) the ratio $\alpha = 1$ ($T = 2.6$ MPa) with O-DDA; (e) the ratio $\alpha = 0.7$ ($T = 3.7$ MPa) with I-DDA; (f) the ratio $\alpha = 0.7$ ($T = 3.7$ MPa) with O-DDA; (g) the ratio $\alpha = 0.3$ ($T = 8.7$ MPa) with I-DDA; (h) the ratio $\alpha = 0.3$ ($T = 8.7$ MPa) with O-DDA.

Therefore, it is important and necessary to consider the tensile strength in dynamic slope stability analysis. The slope with different material behavior on

tension, the landslide initiation, and failure models are also different. For the current study, the I-DDA method can be applied to dynamic slope post-failure analysis considering the different tensile behaviors. However, it is still difficult for the quantitative slope stability analysis considering the tensile and shear characteristics simultaneously as more verifications with the tensile failure considered in the real landslide are needed. More slope stability analysis focusing on tensile failure will be studied in future work.

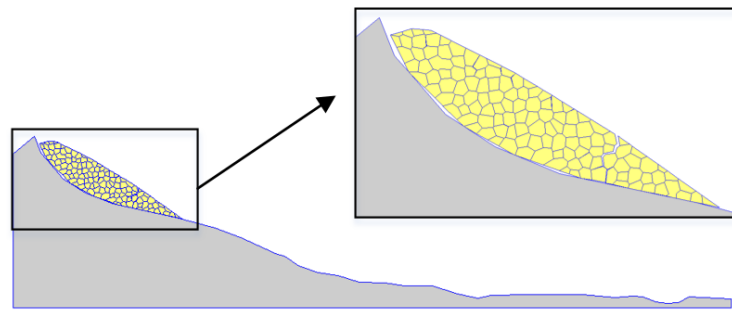
3.7. CONCLUSIONS

In this study, the limitations of the original two-dimensional DDA in detecting the candidate reference for the vertex-to-vertex contact problem were first described. A lost contact case, which will lead to indeterminacy in candidate reference line determination, has been presented. To overcome this defect, a new improved concept based on the contact vector method that is inside or outside the reference line has been proposed and implemented in the 2D-DDA program to improve the accuracy and effectiveness of vertex-to-vertex contact. The new improved concept is a stricter algorithm that can detect the right candidate reference line for vertex-to-vertex contacts in the discovered lost case. The accuracy and effectiveness of the contact in I-DDA have also been verified using various contact examples, including the lost case contact, sliding block contact, blocks contact with convex, and blocks contact with concave. The simulated results have proved that the I-DDA with the new contact algorithm has better performance and accuracy in solving block contact problems in various cases. The improved method can not only be valid for finding the possible reference lines for the vertex-to-vertex contact but also remedy the defect of the lost case, in which the theory of contacts in O-DDA fails. The computational efficiency of the DDA method has been significantly improved, and the I-DDA program can be better applied to block systems with arbitrary vertex-to-vertex contact states.

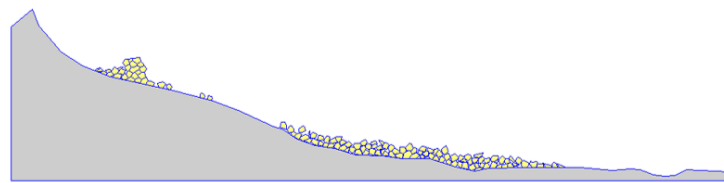
Then, a unified tensile fracture model has been proposed for the two-dimensional DDA method to simulate the tensile behaviors of various materials. The new model unifies four failure modes, i.e., maximum normal stress criterion,

Tresca criterion, Mohr-Coulomb criterion, and von Mises criterion, for tensile failure in the normal direction. The advanced discretization approach has also been incorporated into the I-DDA method. With improvement, the I-DDA can describe the tensile fracture behavior of various problems, such as rock fragmentation in Brazil disc and dynamic slope failure. Firstly, the direct tensile test is computed using the I-DDA method, and the results from the I-DDA are proved to agree well with the theoretical solutions, which verifies the accuracy and validity of the I-DDA method. To show the capacity of I-DDA in describing the tensile behavior of materials, numerical examples of Brazil disc split tests with a central hole and eccentric hole are studied. The simulated results are consistent with the previous experimental results. By using the I-DDA method, the simulated indirect tensile strength of three different Brazil discs has been obtained as well. A comparative analysis on simulation of Brazil disc split tests with I-DDA and O-DDA is presented, and the I-DDA exhibits better performance. The effect of spring stiffness on simulated crack patterns in Brazil disc split tests have also been investigated with I-DDA, the results suggest that the change of spring stiffness and Young's Modulus is available in an appropriate range. The proposed I-DDA method can be a quantitative analysis approach for the rock fragmentation problems in Brazil disc.

A practical Donghekou landslide in the 2008 Wenchuan earthquake is analyzed to verify the applicability of the I-DDA method considering the tensile behavior of rock mass under dynamic seismic loading. The different slope failure behaviors are observed in cases with different tensile strengths by using the I-DDA method. The results show that the proposed method can analyze the slope failure models with consideration of tensile and shear characteristics simultaneously. Three main cases of failure models affected by the ratio of shear strength to tensile strength can be concluded using the I-DDA method: (1) shear failure controlled; (2) shear failure controlled and tensile failure influenced; (3) tensile failure controlled.

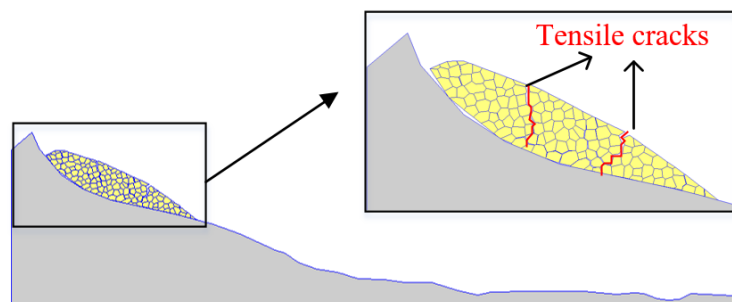


(a)

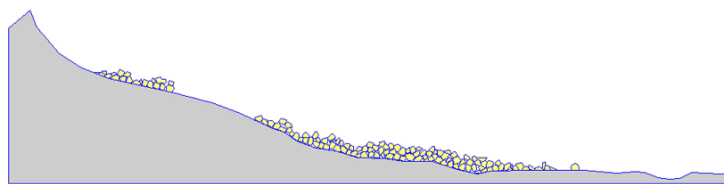


(b)

Figure 3.48 Model of shear failure controlled



(a)



(b)

Figure 3.49 Model of shear failure controlled and tensile failure influenced

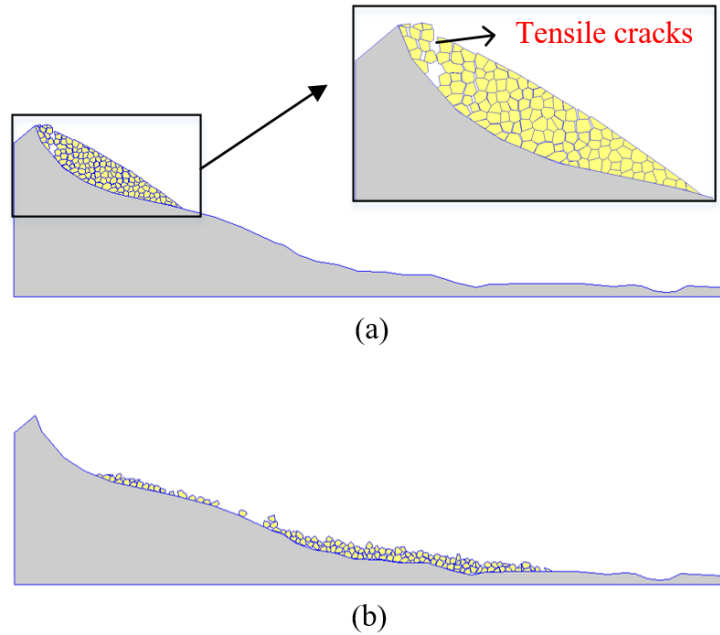


Figure 3.50 Model of tensile failure controlled

As shown in Figure 3.48, the ratio of shear strength to tensile strength is small and the tensile strength of material is relatively stronger than shear strength. In this case, the slope collapsed with shear failure characteristics. Under the seismic ground motion, when the landslide is triggered, the slide mass firstly starts to slide along the base block surface in a whole without obvious tension crack observed in sliding mass. As shown in Figure 3.49, once the landslide is triggered, the rock mass starts to slide and the obvious tensile cracks can be observed. It can be concluded that the landslide is triggered mainly by shear failure while the tensile failure is also observed at the same time. Figure 3.50 shows the model of tensile failure controlled. In this case, the slope behaviors as tensile failure, the tensile cracks normal to sliding surface are always initiated in the sliding mass at first under earthquake, the slope then collapsed and evolved into the landslides.

Both the tensile and shear strength have a significant influence on slope stability analysis. The I-DDA method can be further applied to dynamic fracture problems of various materials in engineering by considering both tensile and shear strength with the unified tensile fracture model. However, to achieve quantitative analysis on the different slope failure models considering both tensile and shear

strength, more experimental and case studies on slope failure should be conducted in future work.

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CHAPTER 4

A NEW METHOD FOR SLOPE STABILITY ANALYSIS UNDER EARTHQUAKES BY CONSIDERING SEISMIC ENERGY

4.1. INTRODUCTION

Over the past decades, many methods have been proposed to analyze earthquake-induced landslides. In general, the methods developed to assess the stability of slopes during earthquakes can be divided into three categories: (1) pseudo-static methods, (2) Newmark's permanent-displacement methods, and (3) stress-strain methods ([Jibson 2011](#)). Terzaghi ([1950](#)) first presented the pseudo-static method, in which the horizontal ground acceleration is applied to simply model the seismic shaking as a permanent body force in the static limit-equilibrium analysis. But it is an oversimplified method for evaluating of seismic stability of a slope. Then, Newmark ([1965](#)) proposed a permanent-displacement method to remedy the weaknesses of the overly simplistic pseudo-static analysis. Since its proposal, Newmark method has been greatly developed and applied for analyzing the slope stability under earthquake ([Wilson and Keefer, 1983](#); [Wartman et al., 2005](#); [Saygili and Rathje 2009](#); [Du 2018](#); [Yiğit 2020](#)). However, both methods of pseudo-static methods and Newmark methods has strength and weakness ([Jibson 2011](#)). Gipprich et al., ([2018](#)) stated that methods of pseudo-static analysis and Newmark's method focus on the effects of earthquake accelerations on the landslide mass, which only accounts for shear failure, but the role of tensile failure is not considered.

With the development of computer techniques and simulation approaches in recent decades, numerical stress-strain methods including finite-difference, distinct-element, and discrete-element modeling, are developed. It is becoming increasingly popular for real earthquake-induced landslide analysis because of its advantage in stress-deformation analysis. DDA is a discontinuous method, which has a great advantage in modeling the deformation and motivation of the fractured and jointed rock masses. Since it is proposed by Shi (1988), the DDA method has been applied extensively for the modeling of the earthquake-induced landslides failure process. The global equilibrium equations of DDA are assembled by minimizing the total potential energy of a discrete system. The mutual interaction between adjacent blocks is represented by contact penalty springs, the non-penetration and non-tension requirements are guaranteed through the open-close iteration process. As DDA has undergone extensive improvements and extensions, it becomes more and more applicable and efficient to dynamic co-seismic landslide analysis (Wu et al., 2004, 2010, 2011, 2017; Chen et al., 2018; Wang et al., 2019; Liu et al., 2019).

However, in the DDA method, the seismic shear failure of joint contacts is determined by a critical value of the static shear strength. Lots of researches on rock mechanics indicate that the materials always show a dynamic property when subjected to dynamic loading (Li et al., 2005; Tetsuji and Tomohiro, 2019; Huang et al., 2011; Li X. F., et al., 2018a, 2018b). The use of the static shear strength for seismic conditions is probably conservative because under short-term loading conditions less creep and relative displacement of the rock and mass will occur in the refuse, resulting in larger short-term stiffness and resistance to loading (Anderson and Kavazanjian, 1995; Zekkos et al., 2007). Latha GM and Garaga A (2010) pointed out that the peak amplitude of a seismic event has no significance on the deformation of slopes during earthquakes from their study on the effect of an acceleration-time response. It is the total that includes amplitude, frequency, and duration of the event that needs to be considered in the analysis. More and more research on the landslide initiation found that the earthquake ground motions with small PGA can also trigger landslide and PGA is not correlated to co-seismic slope displacement (Garini and Gazetas 2013; Song et al. 2016; Chen et al., 2021). The

defect that the dynamic properties of rock mass under earthquake will affect the accuracy of slope stability analyses. The dynamic strength should be considered in the analysis of earthquake-induced landslides.

In recent earthquake-induced landslides in the near-fault region, pulse-like ground motion (PLGM) is regarded as one of the main factors triggering landslides (Gazetas et al. 2009; Garini et al. 2011; Davoodi et al. 2013; Song et al. 2015, 2016; Rodriguez-Marek and Song 2016). It sometimes can cause large-scale landslides even though the peak ground acceleration (PGA) of PLGM is very small. For example, a strong earthquake (M7.3) occurred in Kumamoto prefecture, Japan in 2016, called the Kumamoto earthquake. While the large-scale landslides occurred in the area with smaller PGA and no landslide occurred in the area with larger PGA. The use of the static shear strength in DDA cannot be applied to accurately analyze earthquake-induced landslides with PLGM in the near-fault region. It is obvious that the joint contacts are more likely to fail under the earthquake triggering force with larger PGA if the static shear strength is used in DDA. Although many PGA-based analytical methods have been developed, how to consider seismic energy in slope stability analysis using DDA is a big challenge. A method for evaluating the dynamic failure of slopes considering the seismic energy, especially the large seismic energy in PLGM, is important and necessary to access the earthquake-induced landslide problems.

In this study, to simulate the landslide evaluation process under the dynamic seismic wave considering the energy of seismic waves, especially PLGM in the near-fault region, a method that considers the dynamic strength of slope material has been proposed to analyze earthquake-induced landslide in DDA simulation. This method evaluates the dynamic strength of joint contacts in the DDA method by considering the effect of fatigue and accumulated damage under seismic repeated loadings. Then, Single-Degree-Of-Freedom (SDOF) systems under cyclic loading, PLGM, and Non-PLGM are simulated by proposed DDA to validate the dynamic failure of joint contact. A practical landslide triggered by PLGM in the Kumamoto earthquake is also analyzed to verify the availability and accuracy of the proposed model for dynamic failure simulation.

4.2. TENSILE AND SHEAR FAILURE IN ORIGINAL DDA

In the DDA method, individual blocks are connected and form a block system by joint contacts between blocks and displacement constraints on single blocks. As shown in Figure 4.1, normal and tangential penalty springs are respectively used to join blocks into a block system. The interaction force between adjacent blocks is governed by equations of contact detection and contact treatment. Based on the procedure of contact detection, the contact penalty springs in either the normal or tangent direction of the boundary between two blocks will be added or removed. By adding and removing the contact penalty springs in either the normal or tangent direction of the boundary between two blocks, the blocks can be coupled and separated to simulate shear or tensile failure.

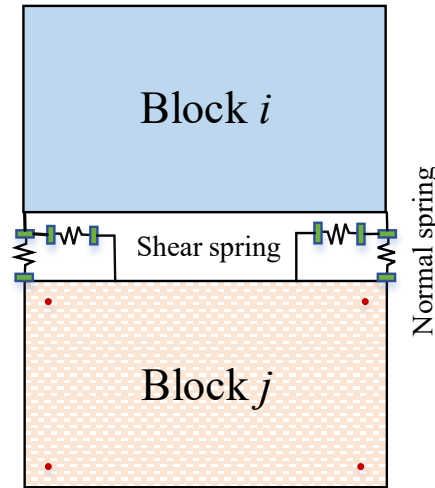


Figure 4.1 Contact penalty springs between adjacent blocks for simulating shear or tensile failure

When the top block is pulled by a loading P in vertical direction while the bottom block is fixed by two fixed points as shown in Figure 4.2, normal extensile stress σ_n will be induced from the springs. When σ_n is less than the tensile strength σ_s of the boundary between two blocks, the deformation of two blocks will occur as a whole since normal springs are used. When σ_n becomes larger than σ_s , the normal penalty springs will be removed so as to allow tension failure to occur, which can be expressed as follows:

$$\sigma_n \geq \sigma_s \quad (4.1)$$

where σ_n is tensile stress induced by loading P and σ_s is tensile strength.

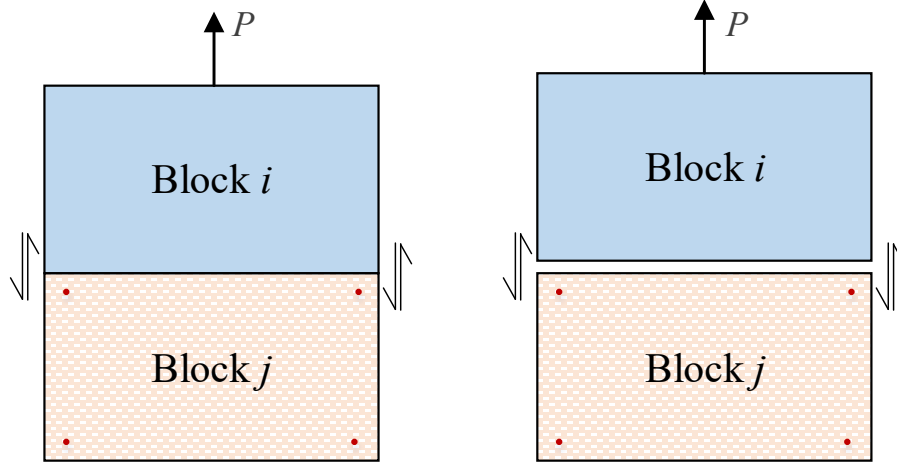


Figure 4.2 Tensile failure in original DDA

Meanwhile, when the top block is pulled by a loading P in horizontal direction while the bottom block is fixed by two fixed points as shown in Figure 4.3, the shear force τ along the boundary will be induced from the deformation of tangent penalty springs. When τ is less than the shearing strength τ_s , the two blocks will be deformed as a whole since tangent penalty springs are used. When τ becomes larger than τ_s , the tangent penalty springs are removed so as to allow shearing failure to occur, which can be expressed as follows.

$$\tau \geq \tau_s \quad (4.2)$$

where τ is shear stress induced by loading P ; τ_s is shear strength, which is estimated by the Mohr-Coulomb's criterion as follows

$$\tau_s = c + \sigma_n \tan \varphi \quad (4.3)$$

where c and φ are the cohesion and friction angle of the boundary between the two adjacent blocks, respectively, σ_n is the normal force from normal penalty springs.

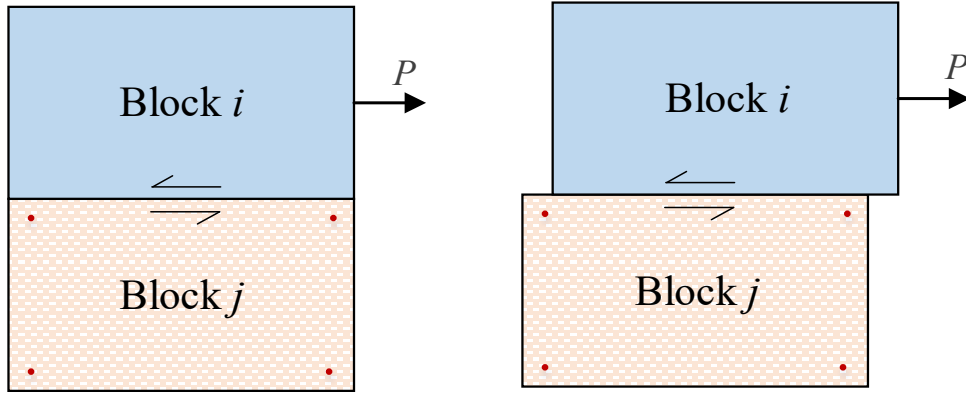


Figure 4.3 Shear failure in the original DDA

In general, the process in contact treatment to realize the satisfaction of the contact conditions at the end of a time step is called open-close iteration (OCI), which includes solving simultaneous equations and the successive checking of interpenetration. After solving the governing equations, an OCI is then implemented to determine the valid system state. The final states of all contacts after solving the governing equations are tested and compared to those assumed before. The system state is considered valid only if all the calculated contact states are consistent with the initial ones. The OCI follows the criteria that the shear spring force cannot exceed the maximum friction, and the distance serves as evidence to determine open and close:

$$k_n d_n > 0 \quad \text{and} \quad k_\tau d_\tau \leq k_n d_n \tan \varphi + \tau_c l \rightarrow \text{locked} \quad (4.4)$$

$$k_n d_n > 0 \quad \text{and} \quad k_\tau d_\tau > k_n d_n \tan \varphi + \tau_c l \rightarrow \text{sliding} \quad (4.5)$$

$$k_n d_n < -\sigma_c l \rightarrow \text{open} \quad (4.6)$$

where τ_c , φ , σ_c and l are cohesion, friction angle, tensile stress and contact length of the contact. k_n and k_τ respectively denotes the stiffness of normal spring and shear spring. d_n is the normal penetration distance and d_τ is the tangential relative displacement.

4.3. A DYNAMIC FAILURE METHOD CONSIDERING SEISMIC ENERGY IN DDA SIMULATION

Earthquake-induced slope failure is related to not only PGA representing the maximum inertial force of a seismic wave but also seismic energy reflecting both amplitudes and frequencies. Although many PGA-based analytical methods have been developed, how to consider seismic energy in slope stability analysis using DDA is a big challenge. Since the energy of a seismic wave is closely related to the amplitudes and frequencies, a dynamic failure method based on the effect of fatigue and cumulative damage of a seismic wave is proposed to analyze earthquake-induced slope failure with the seismic energy considered in DDA simulations. In the following section, the method will be introduced in detail.

Firstly, how to evaluate the dynamic failure of engineering materials under seismic loading is a challenge. Over the past decades, many researches have been conducted to study the dynamic failure characteristics of materials in engineering. To evaluate the dynamic strength of rocks in dynamic response analysis, Okada and Naya (2019) proposed a mathematical model based on a lot of experimental research on the effects of fatigue, loading rate, and cumulative damage. A series of mathematical functions are concluded to quantitatively evaluate dynamic failure. For the effect of fatigue, the cyclic fatigue function f_1 has been determined:

$$f_1(N_f) = \frac{\tau_{f-Nf}}{\tau_{f-N=1}} = 1 - \alpha \log_{10} N_f \quad (4.7)$$

where $N_f (\geq 1)$ is the number of loading cycles at the time of failure, τ_{f-Nf} is the shear strength at the time of failure after loading N_f , $\tau_{f-N=1}$ is the shear strength under monotonic loading ($N = 1$), and α is a parameter defining the slope of the function.

For the effect of cumulative damage, the cyclic damage function f_2 representing the effects of damage owing to cyclic loading before failure can be expressed:

$$f_2(N) = 1 - d(N - 1) \quad (4.8)$$

where d is the slope of the initial and failure points. As the number of the cumulative damage increase, the cyclic damage function is assumed to decrease linearly. Therefore, the linear cumulative damage rule is adopted in this study. The pre-peak

damage of hard rocks exhibits a nonlinear relationship (Eberhardt et al., 1999; Gatelier et al., 2002), the nonlinear cumulative damage rule might be required instead of Eqs. (4.8). However, the linear cumulative damage rule tends to at the side of safe, and it is not an important study for the proposed evaluation method in this research. As the linear cumulative damage rule is used, and then $f_1(N_f) = f_2(N_f)$, therefore the determination of parameter d can be obtained as follows:

$$d = \frac{\alpha \log_{10} N_f}{N_f - 1}, \quad (N_f > 1) \quad (4.9)$$

By submitting the Eq. (4.9) into Eq. (4.8), the residual shear strength ratio under cumulative damage can be obtained. The residual shear strength ratio will decrease as the cumulative damage increases, which can be expressed as:

$$R_{res} = 1 - \frac{\alpha \log_{10} N_f}{N_f - 1} (N - 1), \quad (N_f > 1) \quad (4.10)$$

where R_{res} is the residual shear strength ratio after the N cycles of loading.

As the loading is applied, the residual shear strength ratio will decrease. Once it reaches a particular value such as $R_{res} = 0.7$, as expressed in Tetsuji and Tomohiro (2019) study, suggesting that the failure occurs. However, the mathematical model is based on the cyclic loading tests, which can be applied effectively to evaluate the dynamic strength under cyclic loading. For seismic wave, which is considered as irregular loading with different stress amplitudes and different frequency, the mathematical model is difficult to be applied directivity in numerical simulation analysis.

Therefore, some assumptions for seismic conditions are made. Based on the mathematical model proposed by Tetsuji and Tomohiro (2019), a dynamic failure method considering the effect of fatigue and cumulative damage of a seismic wave is proposed to analyze earthquake-induced slope failure in DDA simulation, which can be expressed as follows:

- (1) An effective-repeated loading stress (ERLS) is defined based on slope's physically static strengths, $\tau_{ERLS} = k_{e1} * \tau_s$ and $\sigma_{ERLS} = k_{e2} * \sigma_s$.

- (2) The corresponding cyclic number of effective-repeated loading (ERL) accumulates as loading stress is larger than ERLS in DDA simulation for the whole seismic wave.
- (3) Once the cyclic number of ERL reaches the critical number N_c , the slope failure occurs.

where τ_{ERLS} and σ_{ERLS} are effective-repeated loading shear and tensile stress in the tangential and normal direction respectively, τ_s and σ_s are the slope's static shear and tensile strengths, k_{e1} and k_{e2} are the coefficients of effective-repeated loading stress in shear and tensile direction, which should be ≥ 0.6 , because only when the loading stress of seismic wave is larger than 60% of the slope's static strengths, the loading stress can be considered as an effective-repeated loading stress (ERLS) being able to cause damage in cyclic loading experiments (Li et al., 2005). When $k_{e1}, k_{e2} \geq 1$, the critical number N_c of effective-repeated loading at failure is assumed to be 1. When $0.6 \leq k_{e1}, k_{e2} < 1$, the critical number N_c of effective-repeated loading at failure is larger than 1, which can be determined based on the dynamic fatigue function in Eq. (4.10). According to this equation, the critical number N_c is related the parameters for dynamic fatigue damage (Okada and Naya, 2019), including the slope of the fatigue function under cyclic loading a , the number of cycles loading at the time of failure N_f and the residual strength ratio R_{res} . Those parameters can be obtained from the experiment data. More experimental studies on this issue will be carried in our future study to improve the accuracy and effectiveness of those parameters in DDA simulation.

With the consideration of the effects of fatigue in the close-open algorithm of DDA, the residual shear strength ratio under irregular seismic loading decreases as the cyclic number of effective-repeated loading (ERL) accumulates, which can be expressed as follows:

$$R_{res} = 1 - \frac{\alpha \log_{10} N_f}{N_f - 1} (N - 1), \quad (N_f > 1) \quad (4.11)$$

where N is the accumulated number of ERL under a seismic wave. Once accumulated number of ERL reaches the critical number N_c , the slope failure occurs.

The determination of parameter N_c can be obtained from the experiment study on material dynamic strength, determined by

$$N_c = \begin{cases} 1 + \frac{(1 - R_{res})(N_f - 1)}{\alpha \log_{10} N_f}, & (N_f > 1) \\ 1, & (N_f = 1) \end{cases} \quad (4.12)$$

As the tensile fatigue test shows a similar fatigue function with the shear fatigue test (Mitsuru Saito, 1984). The function of the residual shear strength ratio is also applied for tensile fatigue failure in DDA simulation, while the parameters a , N_f , R_{res} should be obtained based on the experiment data of the dynamic fatigue test.

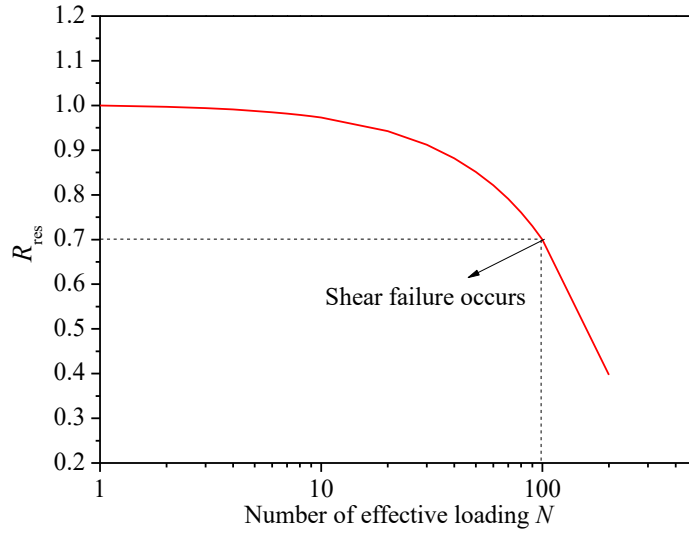


Figure 4.4 Calculation example of the residual shear strength ratio with the number of effective impact N

Figure 4.4 shows a calculation example of the relationship between the residual shear strength ratio and the accumulated number of effective impact N for the case when $a = 0.15$ in Eq. (4.11). The residual shear strength ratio decreases with the increase of the accumulated number of effective impact N when subjected to the dynamic loadings. In this case, when $a = 0.15$, as the accumulated number of effective impact N reaches 100 times, the residual shear strength ratio decreases to 0.7, which means the value of the residual shear strength ratio R_{res} at the assumed dynamic shear failure state and that failure occurs. The parameters a , N_f and R_{res}

can be obtained from the dynamic experiment on rock properties. In this study, we focus on the dynamic failure model in the DDA method.

4.4. VALIDATION EXAMPLES

4.4.1. SDOF MODEL

Single-Degree-of-Freedom (SDOF) system is a fundamental case in structural dynamics. Therefore, the SDOF system is applied to validate the dynamic shear and tensile failure evaluation of the proposed DDA method.

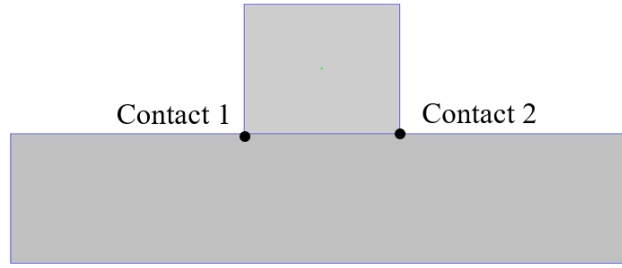
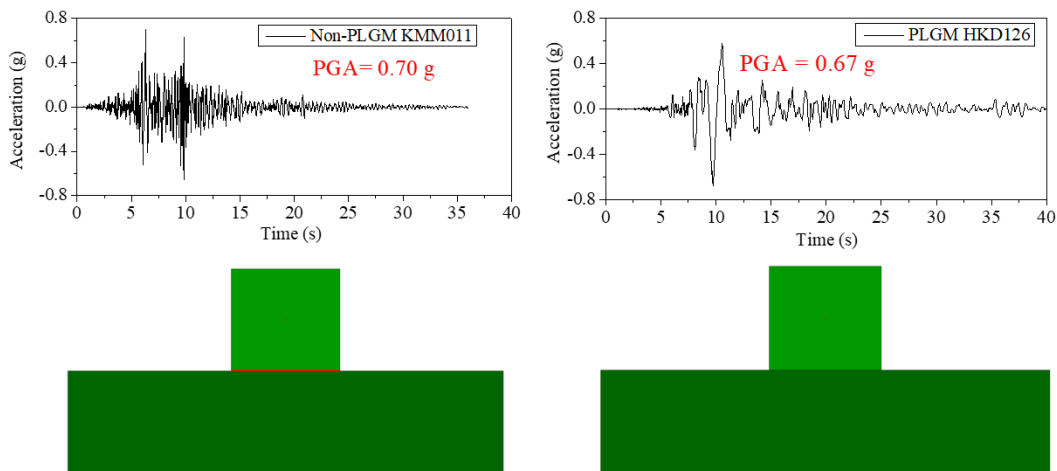


Figure 4.5 Model of SDOF system

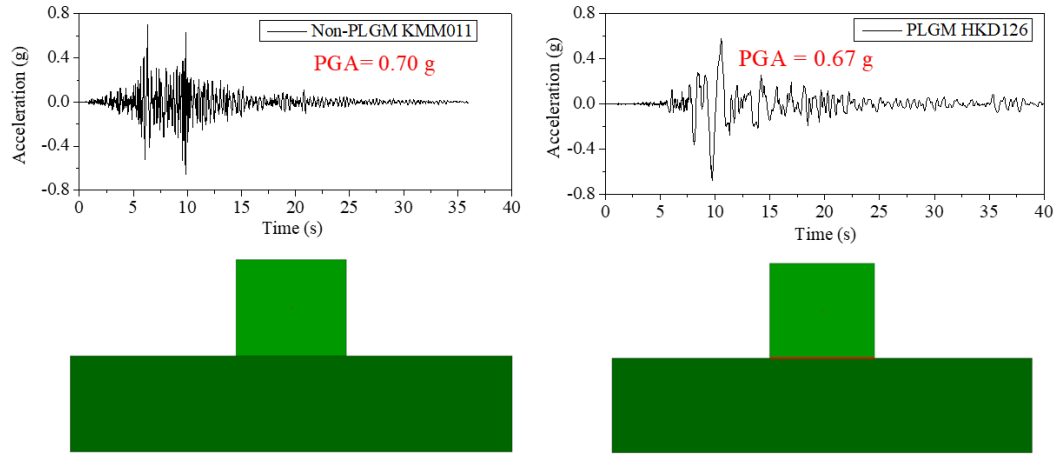
As shown in Figure 4.5, an SDOF model that a square block rests over a bigger base block is constructed. The joint contacts that hold the two blocks together can be separated under the seismic wave loading force. The seismic waves are respectively applied in the horizontal and vertical direction of the bottom base block to simulate the dynamic shear and tensile failure. To validate the effective consideration of fatigue in the proposed DDA method, the SDOF model is calculated with both the original DDA and the proposed DDA method. The joint parameters are as follows: $c = 17142$ Pa, $\varphi = 0^\circ$, $T = 1$ MPa. The coefficients of effective-repeated loading stress in shear and tensile direction $k_{e1} = k_{e2} = 0.6$. The residual strength ratio at the critical failure state is assumed to be 0.7, namely $R_{res} = 0.7$. The dynamic failure parameters for fatigue effect are as follows: $a = 0.1$, $N_f = 100$.

Two seismic waves that one is PLGM with PGA of 0.67g and the other is Non-PLGM with PGA of 0.7g are applied in this simulation, introduced in the previous section (Figure 1.7). In the original DDA, the joint contact failure between two

blocks occurs when the shear force is larger than the peak shear strength, suggesting that the peak value of PGA of a seismic wave dominates the failure behavior of structures in seismic response analysis. As shown in Figure 4.6 (a), the top block of the SDOF model separates from the bottom block when subjected to Non-PLGM while it can remain stable under PLGM because the Non-PLGM has a larger PGA. While in the I-DDA with dynamic failure model, the two blocks separate only when it is subjected to PLGM while it stays stable under Non-PLGM, as shown in Figure 4.6 (b). With the dynamic failure model incorporated, the proposed DDA method can not only consider the peak amplitude of a seismic wave but also take the frequency and duration of a seismic wave into account. The simulated results show that the effect of fatigue of dynamic seismic waves has been effectively considered in the proposed DDA method, which can be better applied to evaluate the dynamic failure in seismic response analysis.



(a) Results of the O-DDA



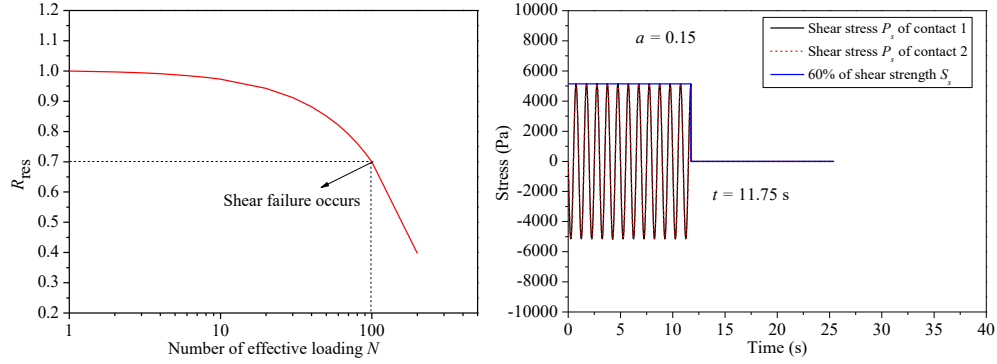
(b) Results of the I-DDA

Figure 4.6 Results of SDOF model subjected to PLGM and Non-PLGM: (a) Results of the O-DDA; (b) Results of the I-DDA

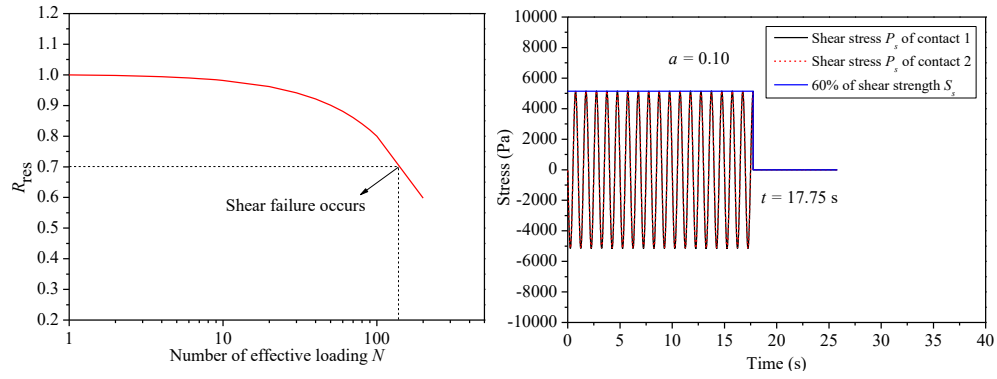
To quantitatively verify the accuracy of the model, the SDOF is further simulated with cyclic loadings. To be specific, a cyclic loading with an amplitude of 0.42g is applied to quantitatively evaluate the dynamic failure of contact joints. The joint parameters are the same as those in the previous simulation. From the previously simulated results of the original DDA, the critical PGA value of seismic ground motion that induces joint failure is 0.7g when the SDOF model has shear strength parameters of $c = 17142$ and $\varphi = 0^\circ$. For the I-DDA simulation considering dynamic strength, the critical PGA value of the ground motions that can cause joint failure is 0.42g because the impact loading is large enough to cause damage to joint contact only if the loading stress is larger than 60% of the static strength. Thus, the cyclic loading with an amplitude of 0.42g is simulated for verification with the I-DDA. The residual strength ratio at critical failure state $R_{res} = 0.7$. Three cases with different dynamic failure parameters for the dynamic fatigue effect are simulated, expressed as: (1) $a = 0.15$, $N_f = 100$; (2) $a = 0.1$, $N_f = 100$; (3) $a = 0.05$, $N_f = 100$.

As shown in Figure 4.7, the simulation results showed that the joint contacts between two blocks all failed under the cyclic loading with an amplitude of 0.42g. As the parameters a decreases from 0.15 to 0.05, the loading time at critical contact failure increases from 11.75 s to 35.26 s since the smaller value of parameter a means that a larger number of effective impact N is necessary, as indicated in Eq.

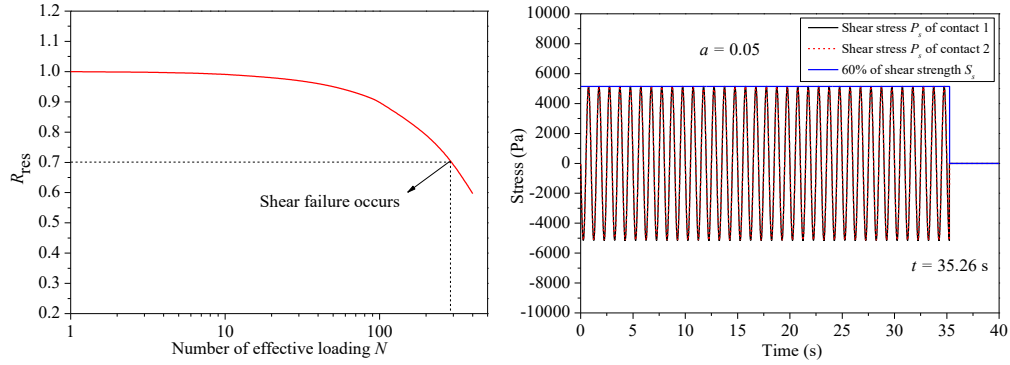
(4.11). The simulation results are in well agreement with the mathematical model for dynamic failure evaluation.



(a) $a = 0.15$



(b) $a = 0.10$



(c) $a = 0.05$

Figure 4.7 Results of SDOF model subjected to cyclic loading

Meanwhile, both PLGM and Non-PLGM are applied to the SDOF model in the vertical direction to verify the dynamic tensile failure. The gravity of the upper block is not considered in the simulation to exclude the effect of gravity on the tensile stress. The tensile strengths are set for contact blocks to resist the tensile

stress from those two PLGM and Non-PLGM respectively. By multiple calculations, as shown in Figure 4.8, the critical tensile strengths to resist the PLGM with PGA of 0.67 g and Non-PLGM with PGA of 0.70 g are calculated as 17.350 kPa for PLGM and 12.362 kPa for Non-PLGM respectively. The simulation results indicate that a larger critical tensile strength is necessary for the model under PLGM than Non-PLGM in this tensile failure simulation when both ground motions have approximately equal values of PGA. The mathematical model for dynamic tensile failure in the normal direction is effectively incorporated in the proposed DDA method.

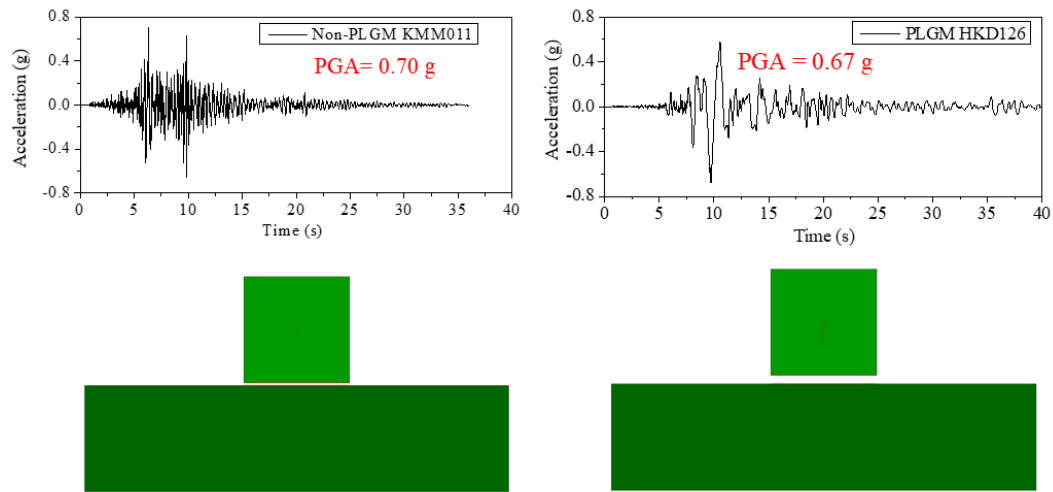


Figure 4.8 Simulation results of dynamic tensile failure

4.4.2. PRACTICAL LANDSLIDE MODEL

To verify the availability and effectiveness of the proposed DDA method for earthquake-induced landslide analysis considering the energy of seismic waves, a practical landslide model is applied for earthquake-induced landslides using the proposed DDA method. As shown in Figure 3.44, this model is constructed based on the longitudinal cross-section of the Donghekou landslide, including two parts: the base block and sliding mass. The sliding mass is randomly discretized into multiple smaller discrete deformable blocks by the Voronoi method. The practical model is simulated using both the proposed DDA method and O-DDA under two cases of seismic waves:

Case (A): PLGM with large energy but a small PGA as shown in Figure 4.9;

Case (B): Non-PLGM with a large PGA but small energy as shown in Figure 4.10.

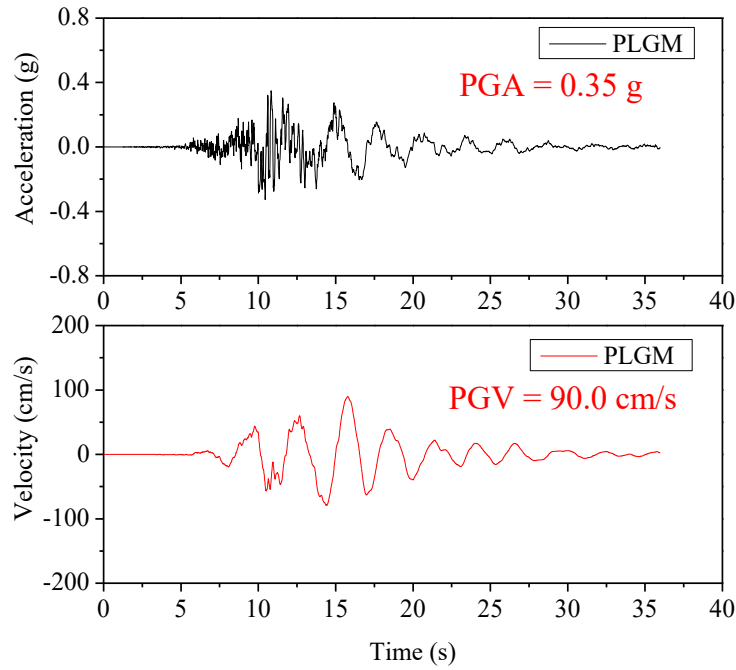


Figure 4.9 PLGM with small PGA for case A

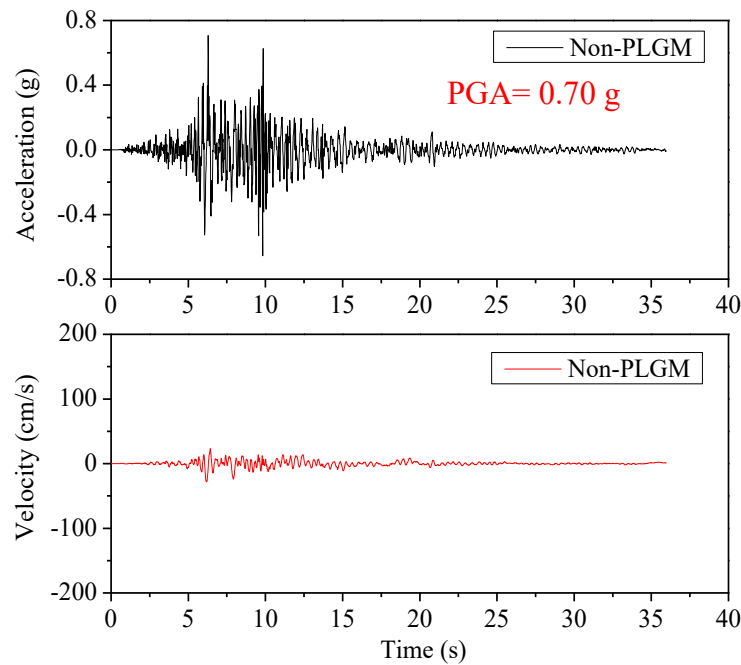


Figure 4.10 Non-PLGM with large PGA for case B

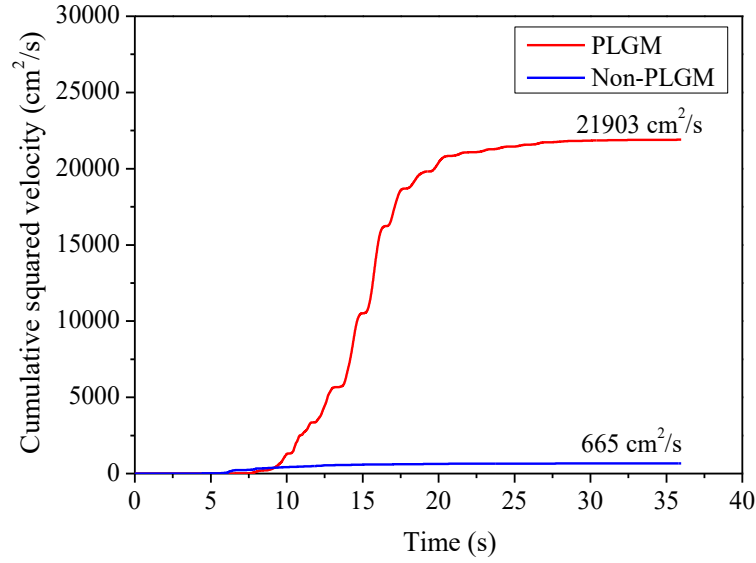


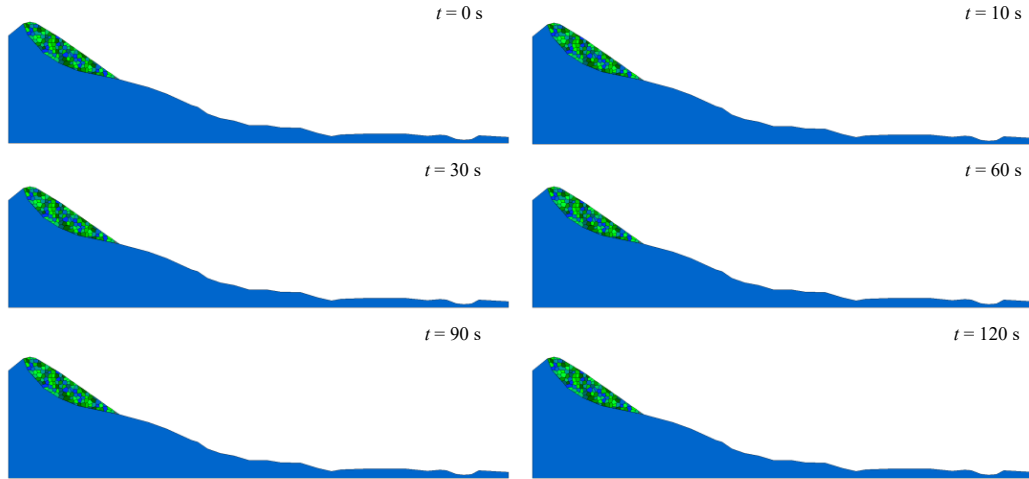
Figure 4.11 Energy distribution of PLGM and Non-PLGM

As introduced in the previous section 1.1.3, the energy of seismic waves can be represented by calculating the CSV. Figure 4.11 shows the energy distribution of two seismic waves in case (A) and (B), it can be seen that the maximum CSV value of PLGM is 21903 cm²/s while the maximum CSV value of Non-PLGM is 665 cm²/s. The energy of PLGM is significantly greater than the energy of Non-PLGM.

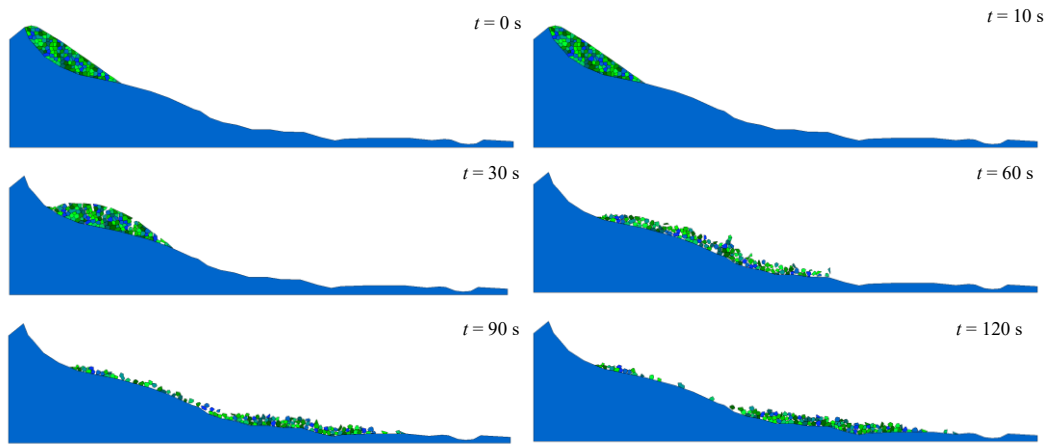
It is clear that the slope internal factors including slope physical characteristics and geometry are also closely related to landslide initiation. However, in this study, we focus on the effect of earthquake trigger forces, especially the difference between the PLGM and Non-PLGM by using the dynamic failure model to verify the effectiveness of the proposed DDA method. The simulation analysis is conducted by using the same model with the same parameters but different seismic waves to exclude the effect of slope internal factors. The material properties for original DDA are taken as follows: density of 2000 kg/m³, Young's modulus of 5.4 GPa, Poisson's ratio of 0.27, unit weight of 20000 N/m³. The cohesion strength of the interface between blocks is 3.2 MPa and the friction angle is 15°.

Figure 4.12 shows the simulated failure process of the practical slope model by using the O-DDA. Compared with the parameters in O-DDA, a set of parameters in the proposed method for dynamic failure are considered. The coefficient of

effective-repeated loading stress is 0.6, denoting that the minimum effective-repeated loading stress which is 60% of slope's physically static strengths is selected in this simulation. The dynamic failure parameters for determining the corresponding critical number of effective-repeated loading based on the fatigue function are taken as: the residual strength ratio at the critical failure state $R_{\text{res}} = 0.7$, the slope of fatigue function $a = 0.05$, Number of loading cycles $N_f = 100$. The material properties for the proposed DDA method are taken as follows: density of 2000 kg/m^3 , Young's modulus of 5.4 GPa , Poisson's ratio of 0.27 , unit weight of 20000 N/m^3 . The cohesion strength of the interface between blocks is 5.0 MPa and the friction angle is 15° . Figure 4.13 shows the simulated failure process of the practical slope model by using the proposed DDA method.

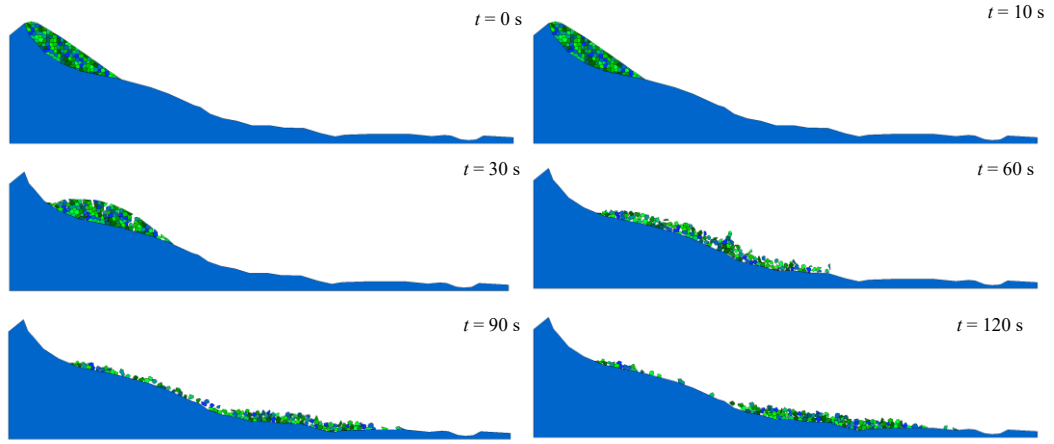


(a) Simulation result of case A from O-DDA

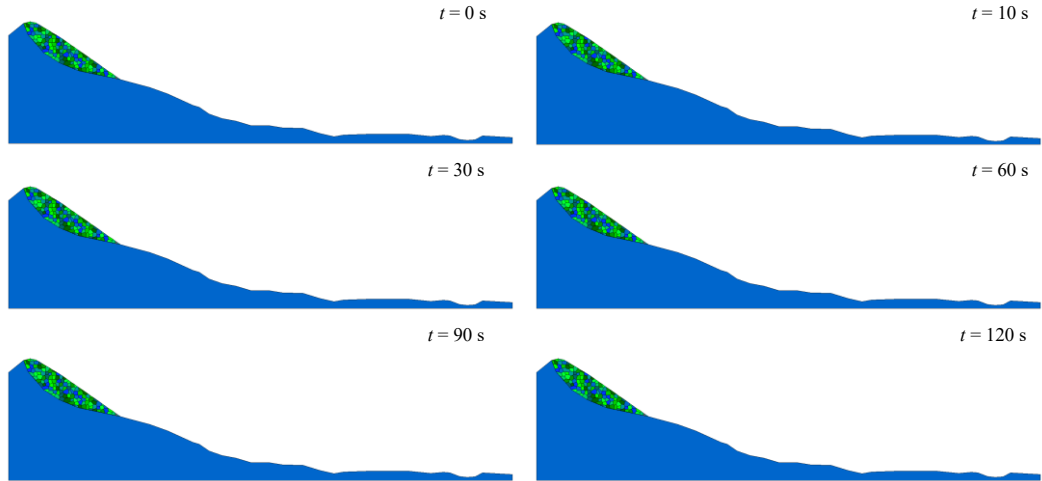


(b) Simulation result of case B from O-DDA

Figure 4.12 Simulation results of slope model from O-DDA



(a) Simulation result of case A from the proposed DDA method



(b) Simulation result of case B from the proposed DDA method

Figure 4.13 Simulation results of slope model from proposed DDA method

As shown in Figure 4.12, the simulation sequences of the slope model under PLGM and Non-PLGM are obtained by using O-DDA. It can be observed that the slope remains stable in the whole process under the PLGM with large energy but small PGA in case A while the slope get collapsed under the Non-PLGM with small energy but large PGA in case B. In the O-DDA, the PGA is considered to be proportional to slope instability for seismic slope stability analysis, the seismic wave of PLGM with large energy cannot be accurately evaluated by using the O-DDA.

While in Figure 4.13, an inversed simulation result can be obtained by using the proposed DDA method. The slope gets failure under the PLGM with large energy but small PGA in case A but it keeps stable under the Non-PLGM with small energy but large PGA in case B. Particularly, in the simulation sequences of slope failure process under the PLGM in case A, the slope is stable before the seismic velocity pulse and starts to slide after the main velocity pulse. The slope instability is proportional to the energy of seismic waves in this simulation by using the proposed method. Therefore, the proposed method is proved to an effective method for slope stability analysis under earthquakes by considering the energy of seismic waves in DDA simulation and the energy of seismic waves is a key triggering factor in slope stability analysis under earthquakes.

In this study, how to take parameters for the dynamic failure model is important for quantitatively evaluating the initiation of earthquake-induced landslide by using the proposed DDA method. Those dynamic failure parameters should be obtained from the experiment. Here, we focus on the effectiveness of the proposed method considering the dynamic strength failure, how to take the parameters for the model is not well considered. More studies for quantitatively evaluating the initiation of earthquake-induced landslides by using the proposed DDA method will be conducted based on experiments and landslide events in our future work.

4.5. CONCLUSIONS

In this study, a method to evaluate the dynamic failure of the slope has been proposed for earthquake-induced landslide analysis by considering the seismic energy in DDA simulation. This method can be applied to effectively evaluate the dynamic strength of joint contacts in the DDA method by considering the effect of fatigue and accumulated damage under seismic dynamic loading. The Single-Degree-Of-Freedom (SDOF) systems under cyclic loading, PLGM, and Non-PLGM have been simulated by using the proposed DDA method and original DDA to validate the accuracy of the proposed model in describing the dynamic failure of contact joints. Meanwhile, a practical landslide model has been simulated under

two cases of seismic waves: case (A) PLGM with large energy but a small PGA; and case (B) Non-PLGM with a large PGA but small energy, using both the proposed DDA method and O-DDA. The results showed that the slope gets failure by using the proposed method while keeps stable by using the original DDA for Case A; but the results have been inversed for Case B, which are also in agreement with seismic response analysis of SDOF system. Thus, the accuracy and effectiveness of the proposed method are validated and the energy of seismic waves is a key triggering factor in slope stability analysis under earthquakes. Compared with O-DDA, the proposed DDA method is effective for simulating the earthquake-induced landslides considering the seismic energy, which is more accurate for earthquake-induced landslides triggered by PLGM.

This study still has space to improve. Here, the aiming of this study is that a method considering the seismic energy for slope stability analysis has been proposed based on the effect of dynamic fatigue failure. The effectiveness of the proposed method has been verified by numerical simulations, how to take the parameters for the model is not considered in this study. For quantitatively evaluating the initiation of earthquake-induced landslide, more experimental studies on the dynamic failure parameters α , N_f and R_{res} should be considered.

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CHAPTER 5

MECHANISM ANALYSIS OF EARTHQUAKE-INDUCED LANDSLIDES FOCUSING ON PULSE-LIKE GROUND MOTIONS ¹

5.1. INTRODUCTION

In this section, the mechanism of earthquake-induced landslides focusing on pulse-like ground motions is investigated based on case studies of earthquake-induced landslide events in the 2016 Kumamoto earthquake and the 2018 Hokkaido earthquake. As described in the previous chapter, many large-scale landslides have been triggered in those two earthquakes, which caused severe damages to infrastructure and lifelines. Lots of researches on earthquake-induced landslide events in those two earthquakes have been conducted to analyze landslide initiation and evaluation (Kazuo et al., 2019; Shinoda et., 2019; Wang et al., 2019; Zhang et al., 2019). However, two phenomena confusing landslide researchers remain unclear. One is that large-scale landslides occurred in the area with small PGA but no landslide in the area with large PGA, for example, the landslides occurred during the 2016 Kumamoto earthquake (Figure 1.2). The other is that one side slopes collapsed while the opposite side slopes back to back remain stable. For example,

¹ Chapter 5 is based on the following journal papers:

Chen, G., **Xia, M.**, Thuy, D. T., & Zhang, Y. (2021). A possible mechanism of earthquake-induced landslides focusing on pulse-like ground motions. *Landslides*, 1-17. <https://doi.org/10.1007/s10346-020-01597-y>

most west-faced slopes collapsed while the east-faced slopes back to back remain stable in the 2018 Hokkaido earthquake (Figure 1.5).

It is believed that slope stability is not only related to the slope inherent factors such as slope shape, geological and physical properties and discontinuities but also affected by the earthquake triggering force. For slopes with similar geological and physical properties, earthquake intensity should be most important factor in estimating the potentially dangerous slopes under earthquakes. Over the past decades, many researchers have paid close attention to intensity parameters responsible for the damage potential of ground motions, which include the peak ground acceleration (PGA), the peak ground velocity (PGV), PGV/PGA, spectral acceleration (S_a), Arias intensity, effective peak acceleration (EPA) and effective peak velocity (EPV) and so on (Malhotra, 1999; Liao et al., 2001; Mavroeidis et al., 2004; Akkar et al., 2005; Tothong and Cornell, 2008). PGA is commonly used as an intensity parameter to represent the damage potential of ground motions in methods of earthquake-induced landslide analysis, such as the pseudo-static method. According to some previous studies, it is clear that the larger the PGA is, the danger the slope becomes. However, the PGA is not correlated well with the landslide initiation in some earthquake events. Especially for the near-fault ground motions, simple peak values of ground motion records, i.e., PGA and PGV, is inappropriate to describe the damage potential of strong ground motions (Yang et al., 2009). Therefore, the method of earthquake-induced landslide based on those simple peak values of ground motion records is unreasonable in those cases with near-fault ground motions.

In this study, energy large enough is believed as a necessary and sufficient condition for landslides to occur. The energy of ground motions is an important intensity parameter, which depends on both amplitudes and frequencies of ground motions. Even an acceleration wave has a large PGA, the velocity of the ground motion could not be large if it is of a high frequency. And then, kinetic energy could also not be large enough for landslide initiation. From the view of energy, we focus our study on the PLGM (Baker, 2007; Zhai et al., 2013; Shahi and Baker, 2014; Chang et al., 2016). PLGM contains large energy even PGA is small in the seismic

wave. Also, not like PGA, PLGM is of directivity, which could be related to the aspect of collapsed slope. Recently, PLGM is of special concern in engineering and has been studied extensively in the architecture field (Malhotra, 1999; Tothong and Cornell, 2008; Jampole et al., 2018; Lu et al., 2018). It has been reported that the large PLGM wave is strongly related to structural damage because the energy in PLGM wave is much larger than Non-PLGM. In recent, some researches for assessing the sliding displacements of the slope have also taken account of the particular characteristics of near-fault pulse-like ground motions and their effects on slope-displacements (Gazetas et al., 2009; Garini et al., 2011, 2013; Davoodi et al., 2013; Song et al., 2015, 2016; Rodriguez and Song, 2016). Displacement-based probabilistic seismic demand analyses considering the PLGM has been proposed and greatly improved. Their researches showed that it is important to consider the long-period and high-amplitude velocity pulse of ground motions in the near-fault region. For PLGMs, the PGV has been proved to be correlated to the seismic slope displacements. The effect of the velocity pulse of PLGM should be considered in co-seismic slope stability analysis.

To study the mechanism of earthquake-induced landslides and explain the two phenomena in near-fault earthquake-induced landslides, the DDA method is applied to landslide simulation focusing on PLGM. At first, the methods for identifying PLGM are reviewed and an effective method is selected for this study. And then, the confusing phenomenon (1) appeared in the 2016 Kumamoto earthquake is studied. The Aso Bridge landslide is analyzed by using the proposed method. Finally, the confusing phenomenon (2) is investigated based on a symmetrical DDA model of two back-to-back slopes. A new possible mechanism of earthquake-induced landslides based on PLGM is proposed to explain those mentioned above two confusing phenomena of earthquake-induced landslides.

5.2. PULSE-LIKE GROUND MOTION (PLGM)

PLGM is defined as the seismic velocity waveform with large amplitude and long duration. Some seismic waves contain PLGM, which is called PLGM seismic waves, while some do not contain PLGM, which is called Non-PLGM seismic

waves in this study. An example is shown in Figure 5.1, in which the seismic waves were recorded during the 2016 Kumamoto earthquake (M7.3). A large pulse velocity is identified in Figure 5.1 (a), but no pulse velocity can be identified in Figure 5.1 (b).

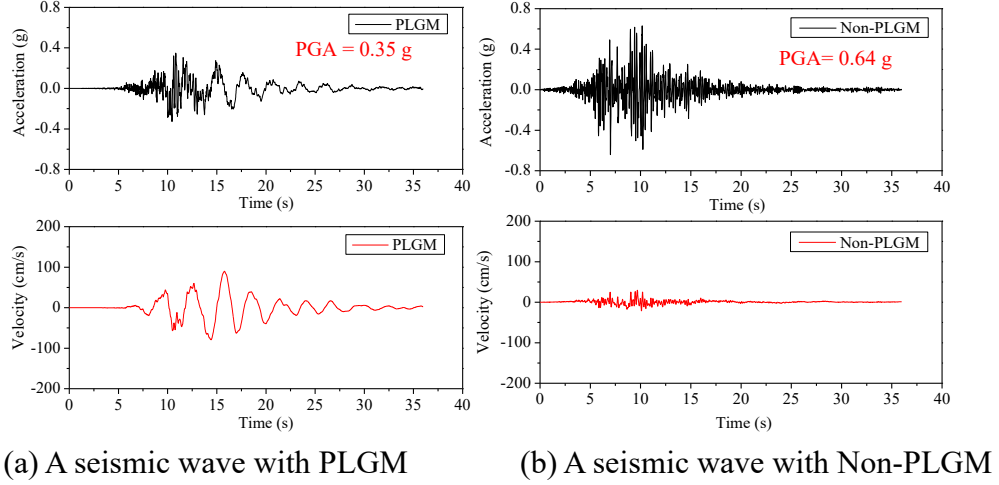


Figure 5.1 An example of PLGM and Non-PLGM seismic waves

The research on pulse-like ground motion can date back to the 1966 Parkfield and the 1971 San Fernando earthquake when a ground motion with great velocity pluses was observed on near-fault region causing severe damage to structure (Mavroeidis and Papageorgiou, 2003). Recently, many researchers have proposed the quantitative classification method of near-fault pulse-like ground motion, which provides a quantitative method for determining the pulse-like ground motions (Baker, 2007; Shahi and Baker, 2013; Zhai et al., 2013). Good knowledge of this classification method can be useful for the representation of PLGM on landslides triggering. Baker (2007) developed a criterion for classifying the near-fault PLGM in the fault-normal direction. Then Shahi and Baker (2013) improved the algorithm for the identification of PLGM on multiple orientations based on Baker's method. Other researchers also proposed many methods for the classification of PLGM (Mavroeidis and Papageorgiou, 2003; Zhai et al., 2013; Chang et al., 2016). In this study, the Baker's algorithm is selected for identification of PLGM (Baker, 2007; Shahi and Baker, 2013). In Baker's method, the three criteria to select pulse-like ground motions are defined as follows:

- (1) The pulse indicator PI, as defined in the following equation, is greater than 0.85.

$$PI = \frac{1}{1 + e^{-23.3 + 14.6(PGV \text{ ratio}) + 20.5(energy \text{ ratio})}} \quad (5.1)$$

where PI is a predictor of the likelihood that a given record is pulselike, taking values between 0 and 1. The large value of PI represents a strong indication that the ground motion is pulselike. Meanwhile, $PGV \text{ ratio} = PGV_{residual} / PGV_{original}$, and $Energy \text{ ratio} = PGV_{residual} / PGV_{original}$.

- (2) The pulse arrives early in the time history, as indicated by $t_{20\%,orig}$ values that are greater than $t_{10\%,pulse}$.
- (3) The original ground motion has a PGV of greater than 30 cm/sec.

Only when the ground motions satisfy all three criteria, it can be identified as PLGM caused by forward directivity effects. As the three criteria for definition of PLGM indicate, the PLGM has properties of large kinetic energy because of the velocity pulses in ground motions. Based on the Baker's method, PLGM can be quantitatively identified and extracted. Meanwhile, the cumulative squared velocity (CSV) is usually calculated to represent the energy of PLGM by the following formula (Baker, 2007; Zhai et al., 2013; Shahi and Baker, 2014; Chang et al., 2016):

$$CSV(t) = \int_0^t V^2(t) dt \quad (5.2)$$

where $CSV(t)$ denotes the cumulative squared velocity at time t and $V(t)$ is the ground-motion velocity at time t .

According to Shahi and Baker's classification method, the velocity pulse can be extracted as shown in Figure 5.2, and the cumulation energy ratio E_r can be obtained by

$$E_r(t) = \frac{\int_0^t v^2(t) dt}{\int_0^\infty v^2(t) dt} \quad (5.3)$$

E_p denotes the pulse-like energy ratio, it can be expressed by

$$E_p = \frac{\int_0^\infty v_{pulse}^2(t) dt}{\int_0^\infty v_{original}^2(t) dt} \quad (5.4)$$

where v_{pulse} is extracted velocity ground motion, $v_{original}$ is original velocity ground motion.

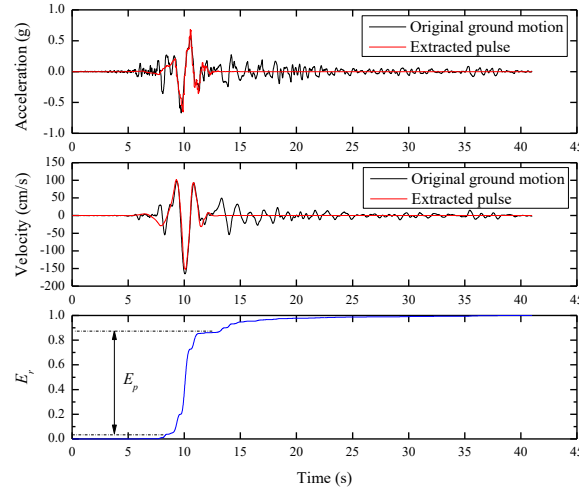


Figure 5.2 Example of pulse-like ground motion extracted by Shahi and Baker's classification method

5.2.1. FORWARD DIRECTIVITY EFFECT

PLGM wave is generated in the area close to a ruptured fault resulting from the effect of so-called forward-directivity of the fault rupture propagation. The forward directivity effect is one of the main causes of PLGM is in the near-fault region. For strike-slip and dip-slip faults, when the direction of fault slip is consistent with the site and the propagation velocity of rupture nearly equals the shear waves' propagation velocity, PLGM is most likely to occur because of the forward directivity effect (Somerville et al., 1997; Baker, 2007). The reason why the forward directivity effect causes PLGM can be explained by the Doppler effect. As rupture propagates from the epicenter, the seismic wave caused by rupture propagation is continuously catching up with the seismic waves generated by previous rupture points. In the forward direction of rupture propagation, velocity waves can be gradually accumulated into a strong velocity pulse, as shown in Figure 5.3.

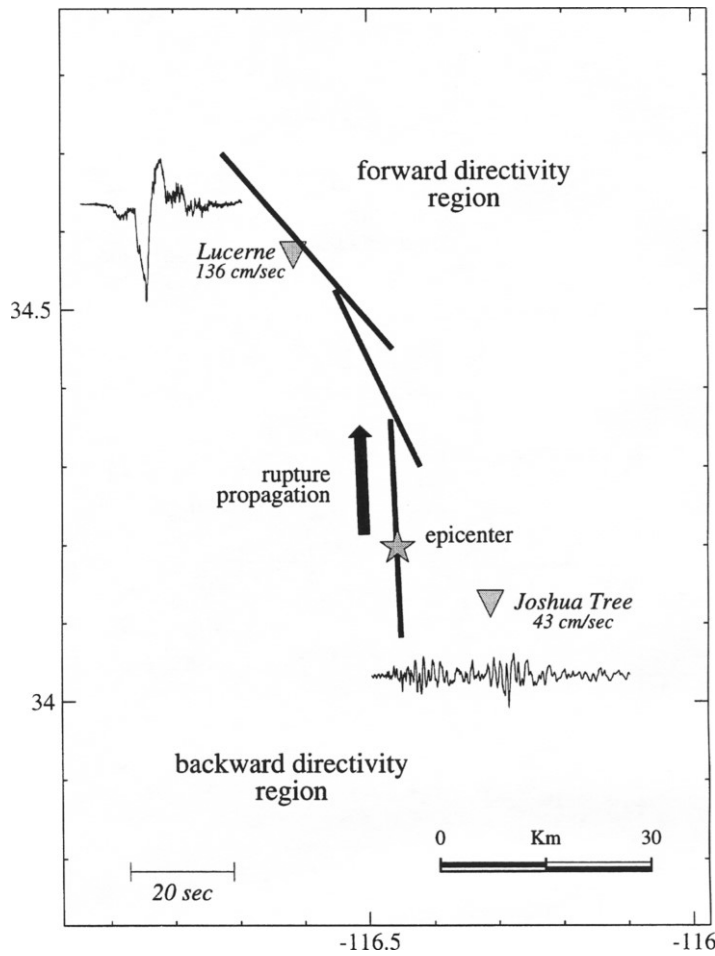


Figure 5.3 Illustration of the forward directivity effect in the 1992 Landers earthquake (Cited from [Somerville et al., 1997](#))

It can also be seen from Figure 1.2 that the large-scale landslide area is located forward-directivity of the right-lateral strike-slip fault in the 2016 Kumamoto earthquake and the PLGM waves were recorded in the stations around the area. Meanwhile, the landslide area in the 2018 Hokkaido earthquake is also located forward-directivity of the reverse slip fault and PLGM is identified in the station HKD128 (Figure 1.4).

Therefore, PLGM is believed to be the possible key trigger factor in clarifying the mechanism of earthquake-induced landslides in this study. Case studies on the earthquake-induced landslides in the 2016 Kumamoto earthquake and the 2018 Hokkaido earthquake are conducted to discuss how PLGM initiates landslides and explains the two phenomena mentioned above. The slope stability analysis is carried out by using DDA simulation with seismic loading.

5.2.2. HISTORICAL EARTHQUAKE EVENTS WITH PLGM IN JAPAN

In Japan, a country with lots of faults and earthquakes, there is great probability of pulse-like ground motions caused in the near-fault earthquakes. In this study, an investigation on observation of PLGM in historical earthquake events has been conducted based on seismic recording data from K-net and Kik-Net, which shows that PLGMs have been frequently observed since 1995 Kobe earthquake by using Shahi and Baker identification method. PLGMs have been observed in many catastrophic earthquake events since 1995 Kobe earthquake. Table 5.1 shows the identified pulse-like ground motions in history since 1995 in Japan. PLGMs has always been observed in those catastrophic earthquake events, the occurrence of the pulse-like ground motion is not a small probability event. Therefore, the study on the mechanism of earthquake-induced landslide with PLGM is important and necessary for earthquake disaster prevention and mitigation, especially in those area at near-fault region.

Table 5.1 Identified pulse-like ground motions in history since 1995 in Japan

Earthquake Name	Year	M_w
Kobe, Japan	1995	6.9
Tottori, Japan	2000	6.6
Niigata, Japan	2004	6.6
Chuetsu-oki, Japan	2007	6.8
Iwate, Japan	2008	6.9
Kumamoto, Japan	2016	7.0
Hokkaido, Japan	2018	6.7
Yamagata, Japan	2019	6.4

5.3. LANDSLIDE SIMULATION USING DDA CONSIDERING SEISMIC LOADING

As we known, lots of landslides are induced during the 2018 Hokkaido earthquake. The slopes are so different from the areas with different ground motions

recorded. As seismic stability of slopes is also controlled by the soil strength, slope geometry and the topography, therefore a symmetrical slope model is adopted for seismic slope stability analysis focusing on PLGMs and Non-PLGMs recorded from the different areas during earthquakes while excluding the effects of the slope geometry and the topography. The parameters of soil strength are also referred to the real values from the actual slopes. As shown in Figure 5.3, the symmetrical slope model is constructed to analyze earthquake-induced landslides using DDA. It consists of two parts: the large block labeled B_1 representing the base slope and 127 small blocks with different shapes generated by the Voronoi method, representing potential sliding mass which is symmetrically distributed on the two sides of the base slope.

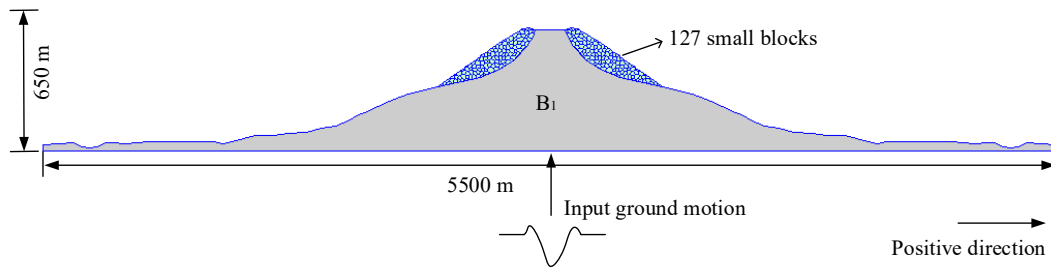


Figure 5.4 A symmetrical landslide model for DDA simulation

The seismic wave is applied at the bottom of the base block in a horizontal direction and the positive value of the input seismic wave is defined as the right direction. The wave propagation in the block system is based on a viscous boundary and the boundary conditions are defined as free at both sides of the base block. Seismic loading is considered as body force in DDA by the product of acceleration and block mass. The seismic body force acts on the base slope block by multiplying the block mass and the acceleration, and it varies with the acceleration waveform. The seismic wave propagates from the base slope block to the potential sliding mass from time to time.

In order to ensure the output acceleration of the base slope block B_1 being the same as the input seismic wave, the mass of B_1 should be large enough. The

acceleration of the whole block system can be calculated based on Newton's second law as follows:

$$M_1 a_s = (M_1 + M_0) a \quad (5.5)$$

where

$$a = \frac{1}{r+1} a_s \quad \text{and} \quad r = \frac{M_0}{M_1} \quad (5.6)$$

where M_1 is the mass of block B₁, M_0 is the mass of the potential sliding blocks, a_s is the input acceleration from seismic wave and a is the acceleration of the whole block system. It is easy to estimate that if $M_1 > 1000 M_0$, the difference between a and a_s will be less than 0.1% from the Eq. (5.6).

An example is shown in Figure 5.5 using the horizontal acceleration from the HKD126 station. The direction of the horizontal acceleration is set to be the horizontal projection of the slope's dip direction. It can be seen that both the acceleration wave and velocity wave calculated from DDA are in good agreement with the input acceleration seismic wave and the corresponding velocity wave. Thus, DDA is applicable to seismic loading-related problems ([Zhang et al., 2013](#)).

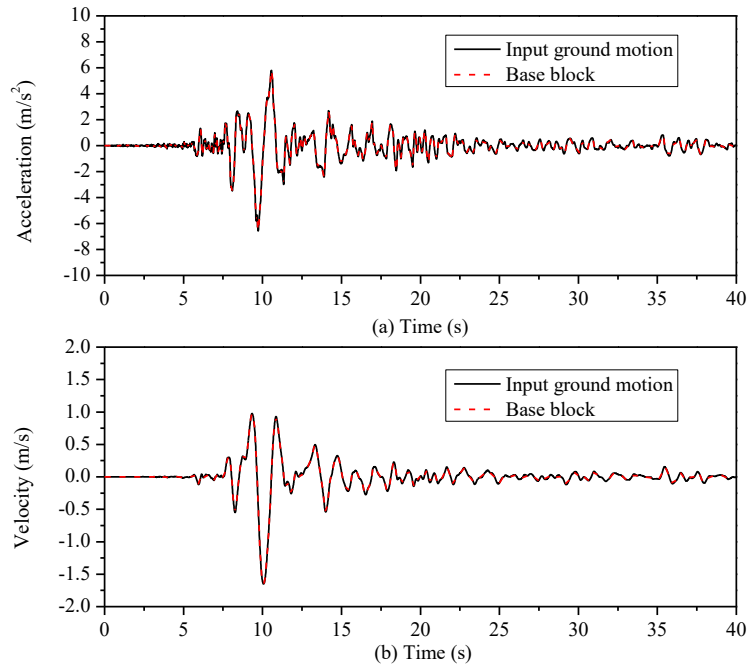


Figure 5.5 Comparison of the input ground motion and the simulation result of the base block using DDA: (a) acceleration and (b) velocity time histories.

5.4. MECHANISM ANALYSIS OF EARTHQUAKE-INDUCED LANDSLIDES

PLGM is a kind of seismic waves with large energy and usually occurs in the area near the seismic fault. In the both the 2016 Kumamoto earthquake and the 2018 Hokkaido Eastern Iburi earthquake, PLGM has been identified near the area where the large-scale landslides with confusing phenomena are observed. In this section, the mechanism of earthquake-induced landslides focusing on PLGM will be clarified by using the proposed DDA method to explain the two confusing phenomena:

The slope model is shown in Figure 5.4 and seismic waves recorded during the Kumamoto earthquake and the Hokkaido earthquake will be used in DDA simulation in order to show the following phenomena

- (1) Why landslides occurred in the area with smaller PGA rather than the area with larger PGA;
- (2) Why one slope collapsed on one side while the other remains stable on the other side for two back-to-back slopes.

5.4.1. LANDSLIDE INITIATED BY PLGM WITH SMALL PGA WHILE LARGE PGA CANNOT

5.4.1.1. CASE STUDY IN THE 2016 KUMAMOTO EARTHQUAKE

In the 2016 Kumamoto earthquake, the phenomenon that landslides occurred in the area where PGAs are small while no landslide occurs in the nearby mountain area where PGAs are large. As shown in

Figure 5.6, three PLGMs are identified in the Aso area where large-scale landslides occurred based on Baker's method for the identification of PLGM, including KMM004, KMM005, and KMM007, and all of them have small PGA. But in the around area where no landslide occurs, the ground motions are confirmed as Non-PLGM even though their PGAs are larger than 0.6 g such as KMM009 and KMM011. To clarify why landslide can be initiated by PLGM with small PGA, a DDA slope model is analyzed by using the energy-based method under all 5 seismic waves in the 2016 Kumamoto earthquake. Those 5 seismic waves can be classified as two sets. One set (set 1) is PLGMs but has small PGA including KMM004, KMM005, and KMM007. The other set (set 2) is Non-PLGMs but have larger PGA including KMM009 and KMM011.

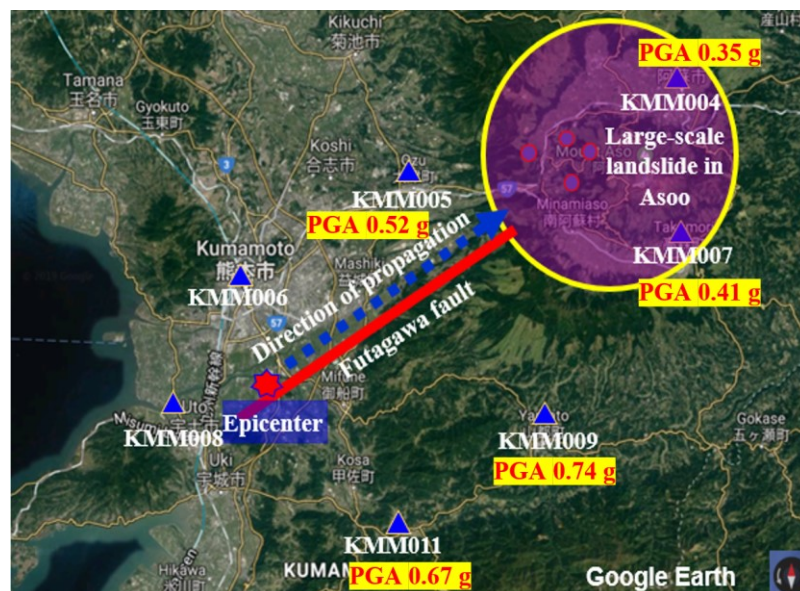


Figure 5.6 The four landslides (circle points) are concentrated at Aso area where PGAs are small while no landslide occurs but PGA (Edited from Google map)

The Aso-Bridge landslide is one of the largest landslides in the 2016 Kumamoto earthquake, which is located on the National Road 57 and destroyed the Aso bridge. It is a large-scale landslide with rock mass that traveled a distance of about 800 m, deposited much debris into the Kurokawa River, as shown in Figure 5.7 (Dang et al., 2016). However, the Aso-Bridge landslide is proved to be triggered by a weak earthquake near station KMM004 where the PGA of recorded ground motion is only 0.35g because it is far from the epicenter. To analyze the landslide initiation and clarify the confusing landslide phenomenon in the 2016 Kumamoto earthquake, the slope model applied in this study is built based on the cross-section of the Aso-Bridge landslide.

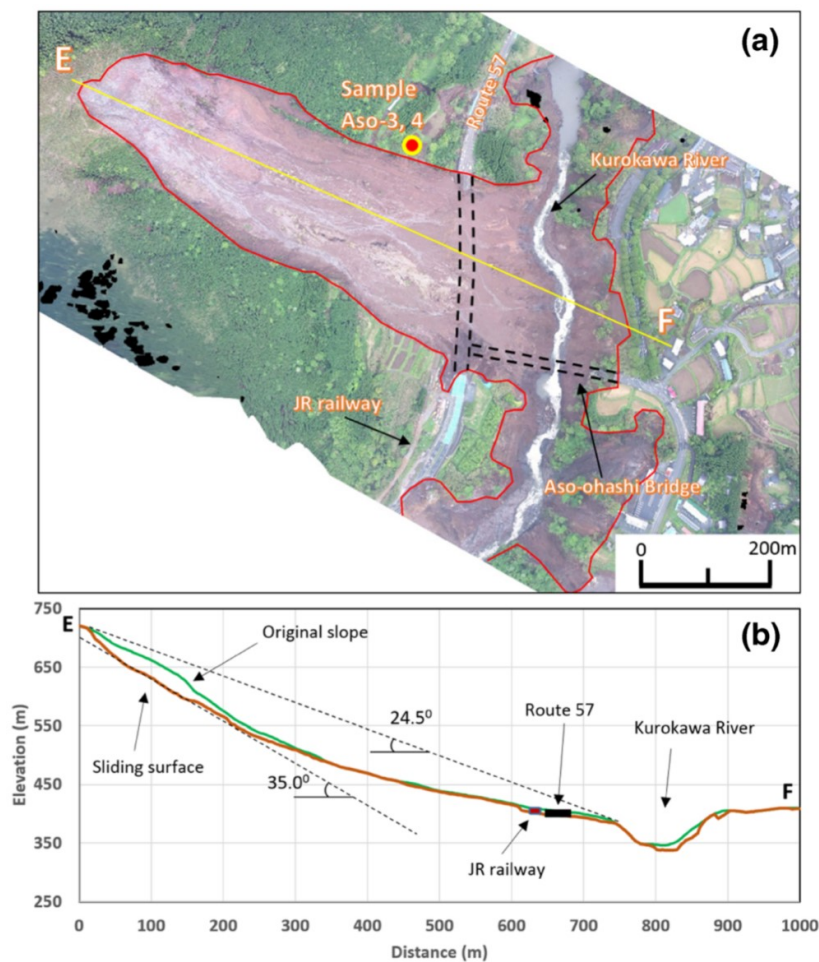


Figure 5.7 Plan (a) and section (b) of the Aso-Bridge landslide (Edited from Dang et al., 2016)

As shown in Figure 5.8, the DDA slope model is constructed based on the longitudinal cross-section of the Aso-Bridge landslide in Figure 5.7 (b), including two parts: the base block and sliding mass. The sliding mass is randomly discretized into multiple smaller discrete deformable blocks by the Voronoi method. In order to analyze the confusing phenomenon (1), two sets of seismic waves are applied as input ground motion in the slope simulation analysis:

- (1) Set 1 are PLGMs from KMM004, KMM005, and KMM007, all the ground motions have large energy but PGAs are small.
- (2) Set 2 are Non-PLGMs from KMM009 and KMM011 where no landslide occurs but PGAs are larger than 0.6 g.

The physical properties are taken based on the Aso-Bridge landslide. According to field investigation on this landslide (Shinoda et al., 2019), the material properties are taken as follows: density of 2150 kg/m^3 , Young's modulus of 7.0 GPa, Poisson's ratio of 0.27, unit weight of 21500 N/m^3 , while the base block has a density of 20000000 kg/m^3 (virtual), Young's modulus of 7.0 GPa, Poisson's ratio of 0.1, unit weight of 0 (virtual). The cohesion strength of the interface between blocks is 0.9 MPa and the friction angle is 20° , tensile strength $T = 0.9 \text{ MPa}$.

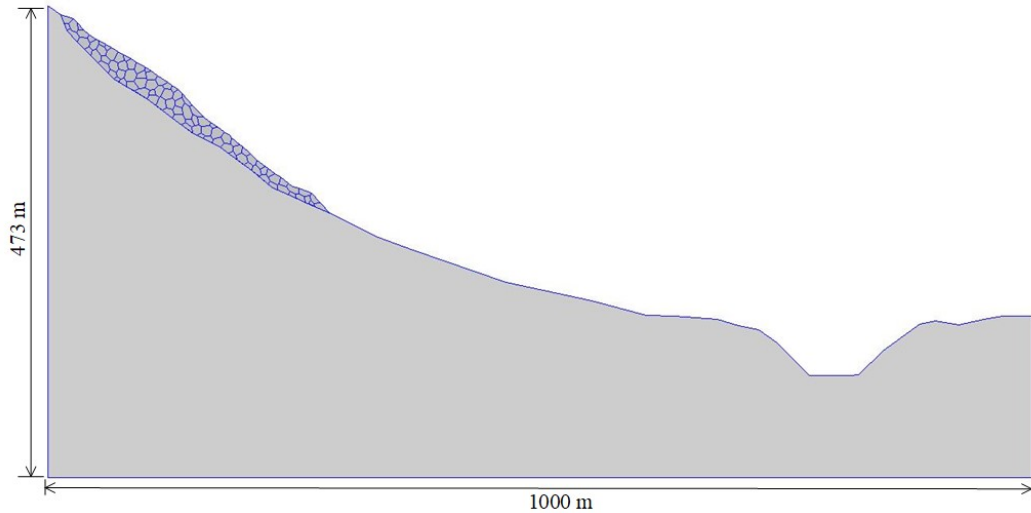


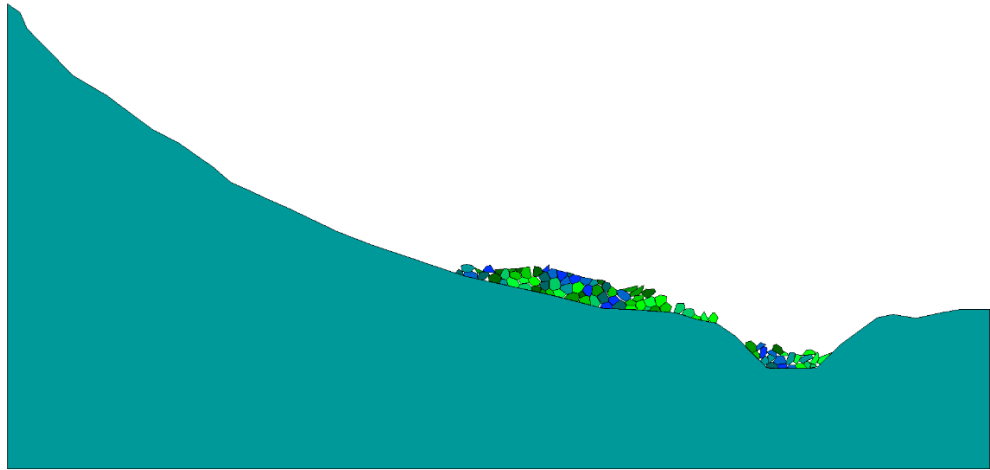
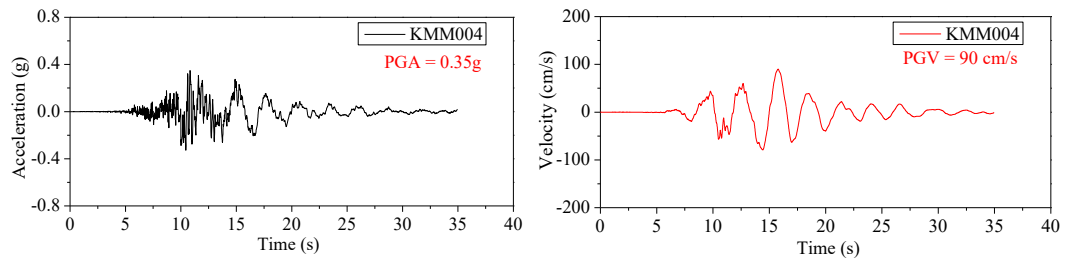
Figure 5.8 Model of Aso-Bridge landslide

The relevant calculation control parameters are as follows: normal contact spring stiffness $k_n = 0.4 \text{ GPa}$, shear contact spring stiffness $k_\tau = 0.4 k_n$, maximum

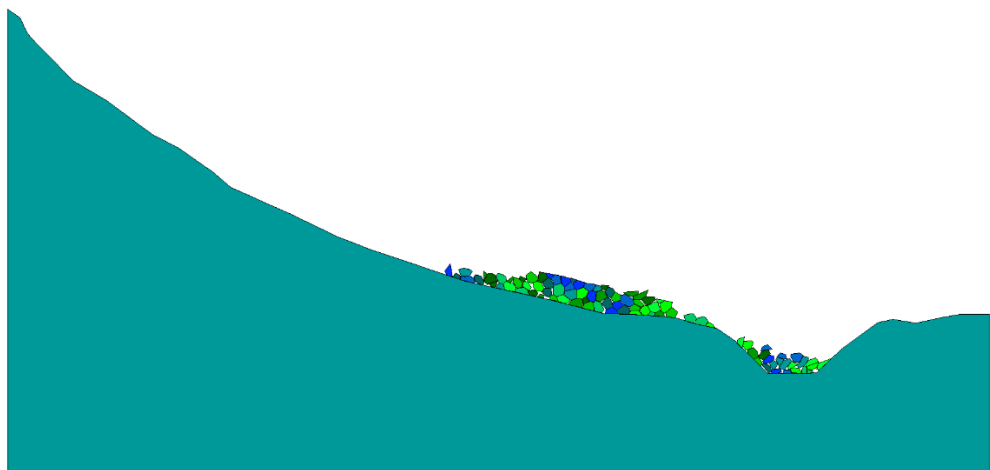
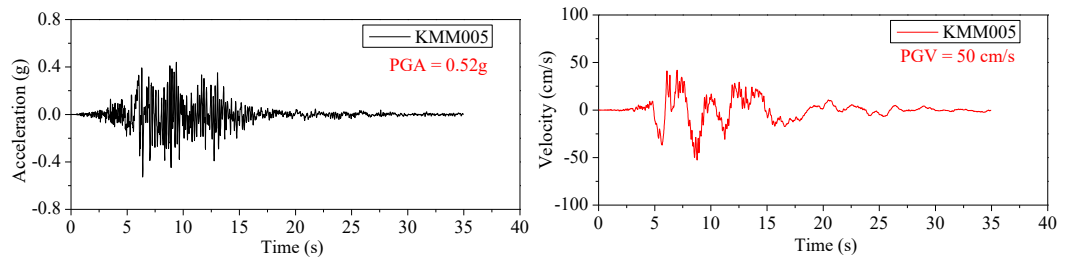
time step size $\Delta t = 0.005$ s, maximum time-step displacement ratio of 0.001, and physical simulation time of 80.0 s. According to the energy-based method, the coefficient of effective-repeated loading stress is $k_{e1} = k_{e2} = 0.7$. The dynamic failure parameters for determining the critical number of effective-repeated loading N_c are as follows: the residual strength ratio at the critical failure state $R_{res} = 0.7$, slope of fatigue function $a = 0.05$, Number of loading cycles $N_f = 100$.

The simulation results are as follows.

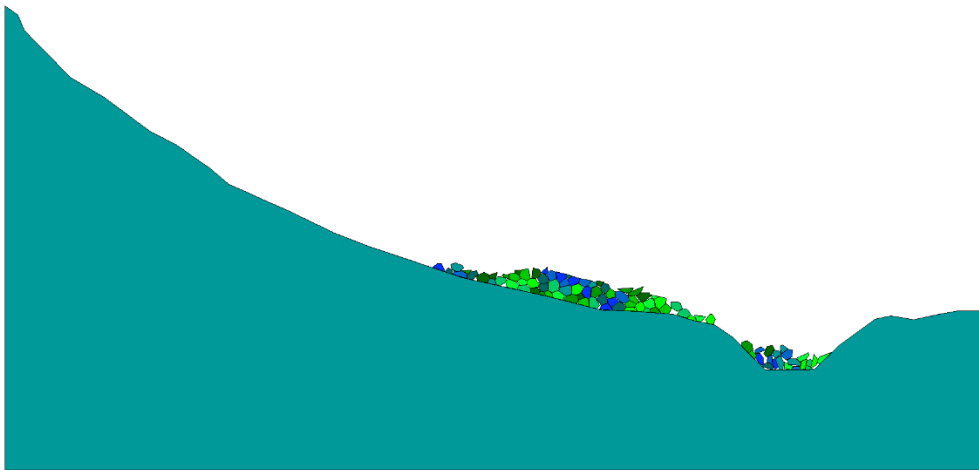
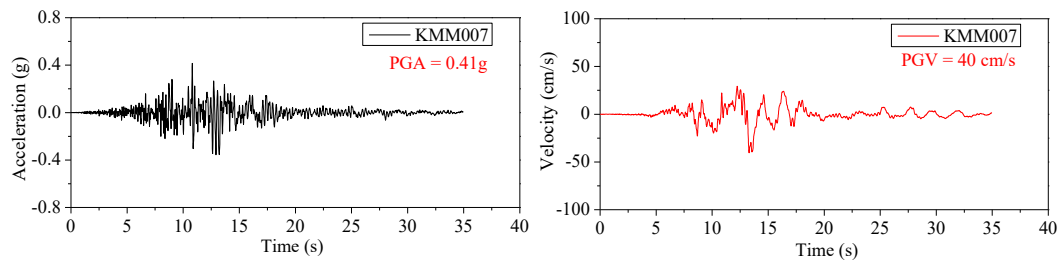
- (1) The slope collapses under the seismic loadings of set 1 with PLGMs although they have smaller PGA values than set 2, as shown in Figure 5.9.
- (2) The slope remains stable under the seismic loadings of set 2 without PLGM, although they have larger PGA values than set 1, as shown in Figure 5.10.



(a) KMM004 ($t = 80$ s)

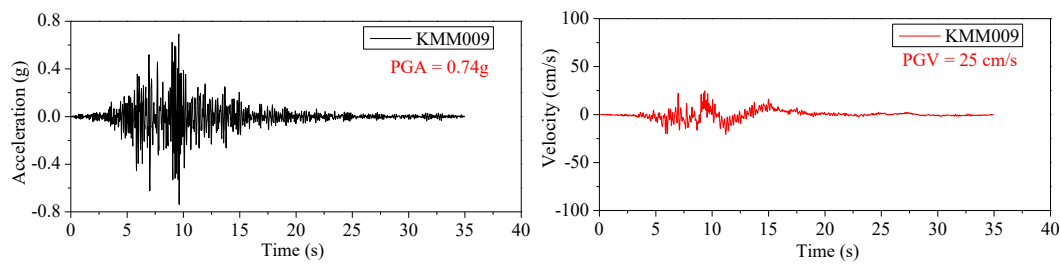


(b) KMM005 ($t = 80$ s)

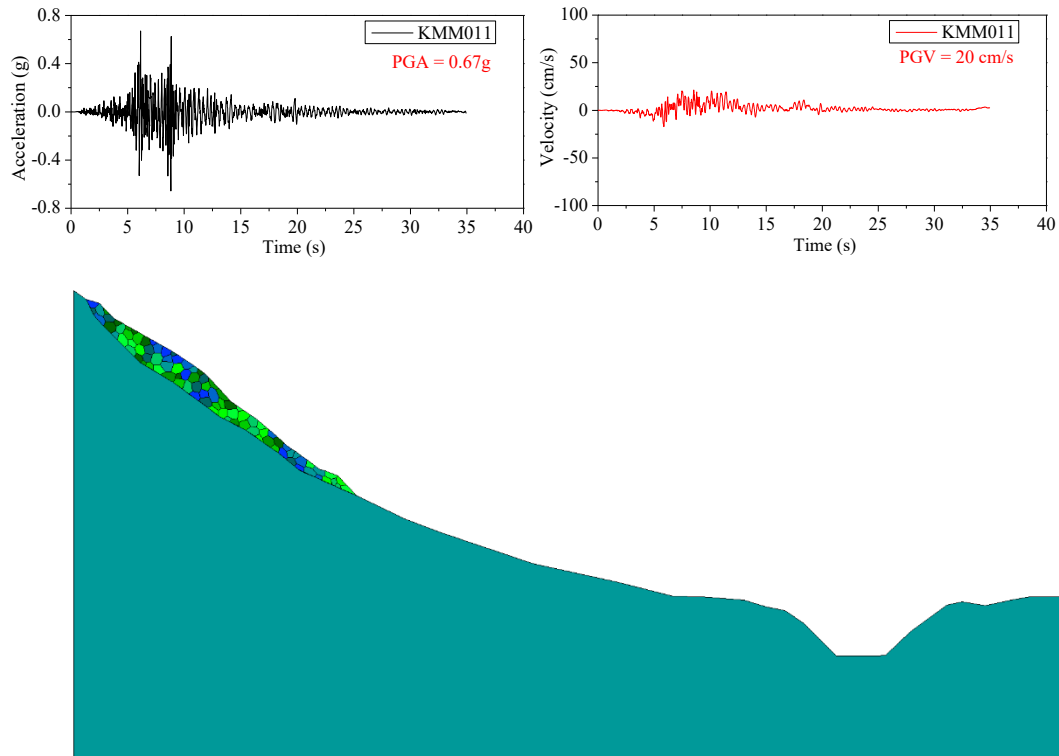


(c) KMM007 ($t = 80$ s)

Figure 5.9 Simulation results of Aso-Bridge slope model under PLGMs



(a) KMM009 ($t = 80$ s)



(b) KMM011 ($t = 80$ s)

Figure 5.10 Simulation results of slope model under Non-PLGMs

It can be seen that the seismic wave with PLGM does induce landslides even its PGA is small while the Non-PLGMs with large PGA does not always induce landslides for the slope with the same strength parameters, which is consistent with the phenomenon that the large-scale landslides occurred in Aso area with so small PGA values while no landslide occurred in the area with larger PGA during 2016 Kumamoto earthquake.

Thus, it is believed that PLGM is one of the significant factors for the mechanism of earthquake-induced landslides. By comparing the velocity waves, it can be seen that a PLGM wave is of larger amplitude and long duration time, i.e., much larger kinetic energy than No-PLGM. That is, the large pulse can provide energy large enough to initiate landslides. If there is no such kind of pulse, there could be no energy large enough to initiate a landslide. The PGA and energy of the above-mentioned seismic waves are listed in Table 5.2. It is obvious that the PLGM

waves have significantly larger energy than the Non-PLGM waves although the PGA values are reversed. Thus, the energy of ground motions should be considered as one of the major factors in slope stability analysis. However, in numerical methods for slope stability analysis under earthquake, the static shear and tensile strength are mostly used. The effect of seismic energy of PLGM is not considered. How to analyze slope stability under earthquakes by considering seismic energy is a big challenge. Therefore, the proposed method fulfills this scope using the discrete element method for slope stability analysis by considering seismic energy. To effectively analyze the slope failure induced by the earthquake, especially for those PLGM with a small PGA but large energy, the proposed method is recommended. The analysis result in this study has proved that it is effective for slope stability analysis by considering seismic energy.

Table 5.2 The recorded ground motions in the 2016 Kumamoto earthquake

Sites	PGA (g)	PGV (cm/s)	Energy (cm ² /s)	PLGM (Yes/No)
KMM004	0.35	90	21895	Yes
KMM005	0.52	50	4751	Yes
KMM007	0.41	40	2148	Yes
KMM009	0.74	25	733	No
KMM011	0.67	20	829	No

5.4.1.2. STUDIES IN THE 2018 HOKKAIDO EARTHQUAKE KUMAMOTO

In the 2018 Hokkaido earthquake, PLGM has also been observed, such as HKD128 and HKD 126. As introduced in section 1.1.1, a large number of landslides have been triggered in the 2018 Hokkaido Eastern Iburu earthquake (M6.7). As shown in Figure 1.4, the landslide areas are observed to be close to the sources of PLGM instead of those with the highest PGA. PLGM is also one of the key factors

of triggering landslides in the 2018 Hokkaido earthquake. The following examples are calculated to further verify this conclusion.

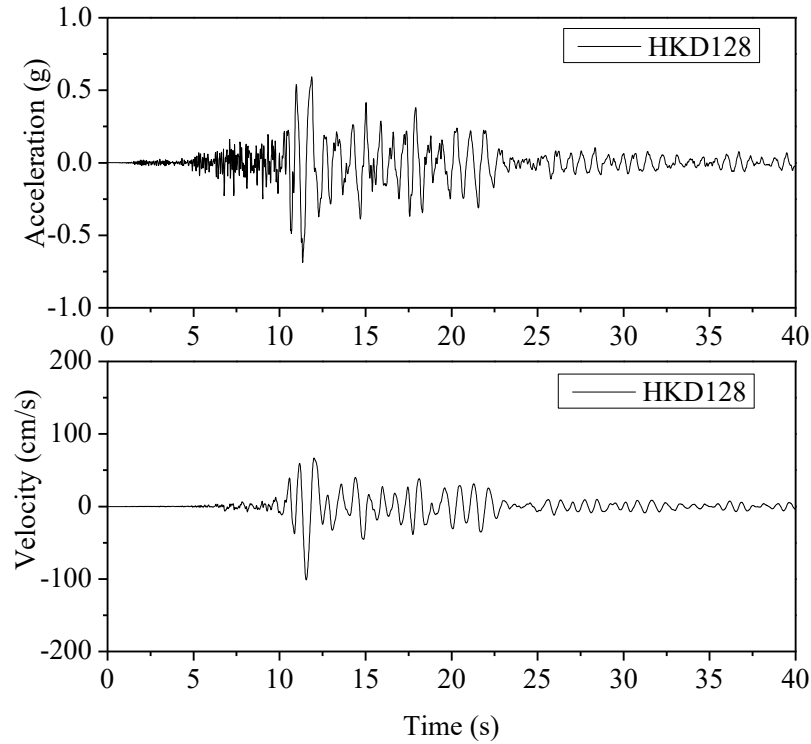


Figure 5.11 Seismic ground motion from HKD128

Figure 5.11 shows the typical PLGM in the 2018 Hokkaido earthquake recorded in the HKD128 monitoring station where a great number of landslides were triggered in this area. Based on Baker's algorithm of the quantitative classification methods ([Baker 2007](#); [Shahi and Baker, 2014](#)), the velocity pulse that contains the main parts of total energy can be extracted from the velocity time history. The original, extracted pulse and residual ground motion of PLGM HKD128 are obtained, respectively, as shown in Figure 5.12.

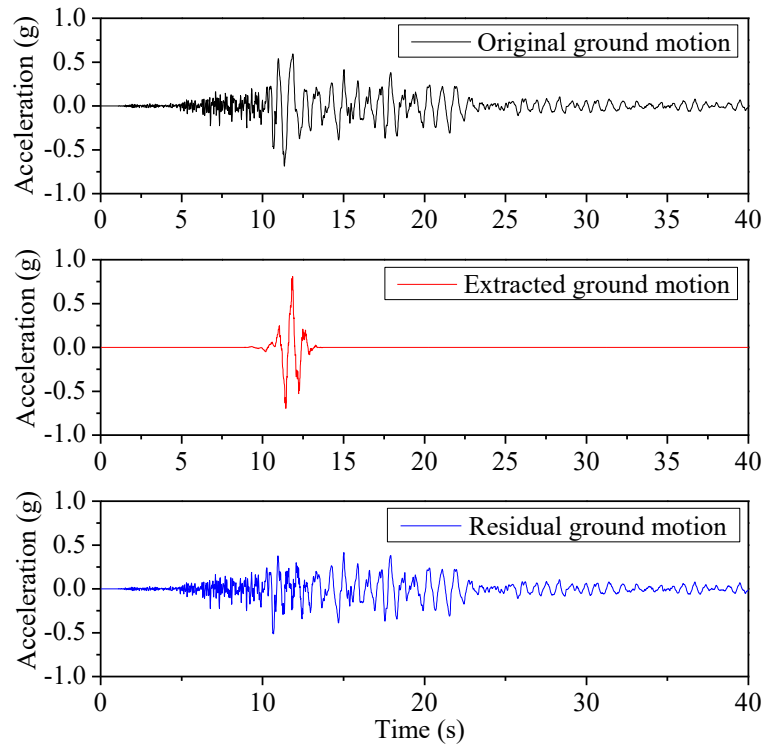


Figure 5.12 Original ground motion, extracted ground motion and residual ground motion of HKD 128

The symmetrical slope subjected to those ground motions is computed using improved DDA. Figure 5.13 compared the simulated result of symmetrical slope under original ground motion, extracted pulse and residual ground motion from station HKD128. It can be seen in Figure 5.13 (a)-(b) that the slope collapse under both original ground motion and extracted original velocity pulse, while the residual ground motion in Figure 5.13 (c) without velocity pulse cannot initiate the landslide even though the PGA is still as strong as the original ground motion, which proves that the landslide initiation is closely related to velocity pulse, the pulse of PLGM which contains the main parts of total energy can be regarded as the major factor initiating the landslide.

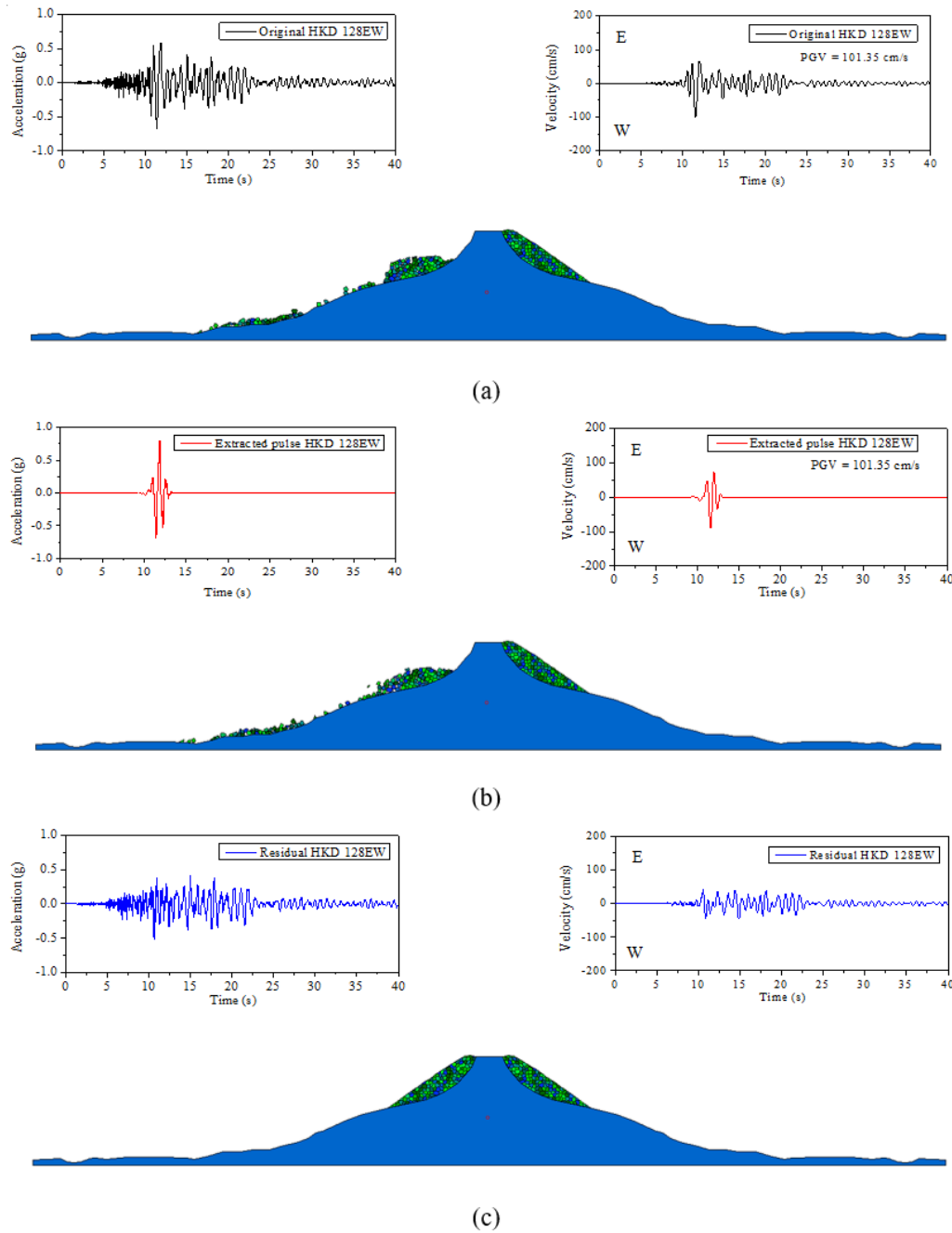


Figure 5.13 Comparison of symmetrical slope under (a) Original ground motion, (b) Extracted pulse, (c) Residual ground motion from station HKD128

In the 2016 Kumamoto earthquake and the 2018 Hokkaido earthquake, the landslide areas are observed to be close to the sources of PLGM instead of those with the highest PGA. PLGM contains much higher energy than Non-PLGM although its PGA is mostly lower. By analyzing the PLGM caused by forward directivity effect and the large energy it contains, the confusing phenomenon that

landslides occurred in the area where PGAs are small while no landslide occurs in nearby mountain area where PGAs are large can be well explained. PLGM is one of key factors triggering landslides in slope failure analysis due to its substantially large energy. The energy of earthquake seismic waves should be considered in slope stability analysis. Especially in the forward directivity effect area near seismic fault, PLGM are frequently observed even though it is far from epicenter. It always has a small PGA but a large energy, which is a big threat to slope stability and can cause catastrophic damage. Slope stability analysis focusing on PLGM which contains large seismic energy should be paid more attention in disaster prevention and mitigation project.

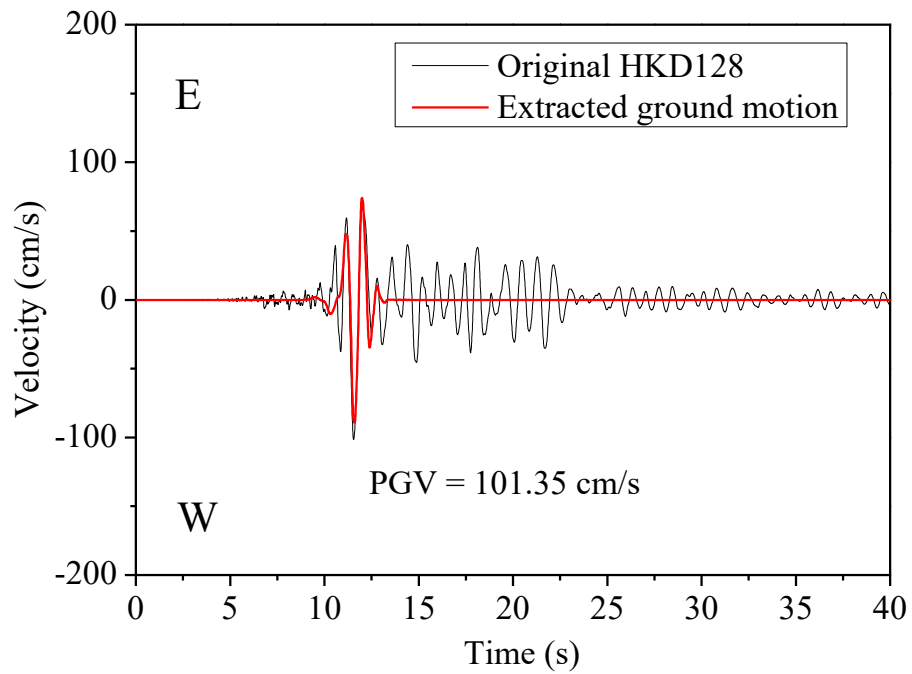
However, the static shear and tensile strength are mostly used in the numerical method for slope stability analysis under earthquake. For example, the static tensile and shear strength are used in original DDA method. Once the shear and tensile stresses are larger than the corresponding resistance in tangential or normal direction, the failure occurs. The slope failure in original DDA is PGA dependent. How to analyze slope stability under earthquake by considering seismic energy is a big challenge. Therefore, the proposed method fulfill this scope using discrete element method for slope stability analysis with seismic energy considered. To analyze the slope failure induced by earthquake which has large energy, especially for those PLGM with a small PGA, the proposed method is recommended. The analysis result in this study has proved that it is effective for slope stability analysis by considering seismic energy.

5.4.2. DIRECTIONAL CHARACTERISTIC ANALYSIS OF COLLAPSED SLOPE

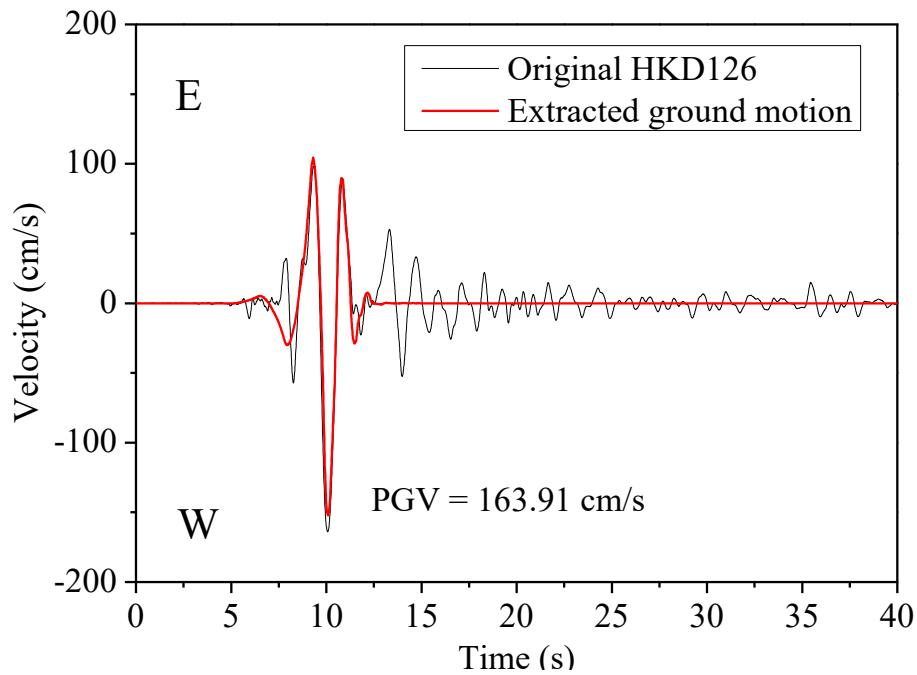
The second confusing phenomenon that the only one side slope failure phenomenon has been frequently observed in the 2018 Hokkaido earthquake. It is found that most collapsed slopes are west-faced while east-faced slopes back to back remain stable. To investigate the relationship between slope failure direction and velocity pulse, PLGM in the 2018 Hokkaido earthquake are extracted and applied respectively on slope model for directional characteristic analysis.

5.4.2.1. ASPECT OF THE COLLAPSED SLOPE IS CONSISTENT WITH THE DIRECTION OF VELOCITY PULSE

Figure 5.14 shows the identified PLGM that initiates the landslides in the Hokkaido earthquake, and the strong westward velocity pulses with PGV over 100 cm/s are observed in those seismic waves HKD128 and HKD126. The two seismic waves will be applied to analyze the relationship between the direction of the collapsed slope and the velocity pulse of PLGM. The model subjected to HKD128 and HKD126 has its parameters as shown in Table 5.3 and Table 5.4, respectively.



(a) HKD128EW



(b) HKD126EW

Figure 5.14 Westward velocity pulse of PLGM at HKD128 and HKD126

Table 5.3 The material and joint parameters of the applied DDA model for HKD128

	Material parameters				Joint parameters		
	Mass (kg/m ³)	Unit weight (N/m ³)	Young's modulus (Pa)	Poisson's ratio	Cohesion (MPa)	Friction angle	Tensile (kPa)
Sliding blocks	2150	-21500	5.4E+9	0.28	3.0	18.7°	250
Base block	2.5E+9	0	5.4E+9	0.1			

Table 5.4 The material and joint parameters of the applied DDA model for HKD126

	Material parameters				Joint parameters		
	Mass (kg/m ³)	Unit weight (N/m ³)	Young's modulus (Pa)	Poisson's ratio	Cohesion (MPa)	Friction angle	Tensile (kPa)

Sliding blocks	2150	-21500	5.4E+9	0.28	4.4	18.7°	15
Base block	2.5E+9	0	5.4E+9	0.1			

As shown in Figure 5.15, it can be seen that the slopes always collapse at the direction of the pulse (DOP) under both ground motions from HKD128 and HKD126. The original ground motion triggers the west side to collapse while the east side of the slope remains stable, which is consistent with the mostly west side slope failure phenomenon in the Hokkaido earthquake. The values of PLGM are reversed to verify the impact of the DOP on the directional feature of the slope. The results show that the collapsed side of the slopes is reversed along with the reversed PLGM. For example, with HKD128 PLGM, the original ground motion triggers the west side to collapse while the reversed PLGM induces the east side of the slope to slide down. Similarly, the same result under PLGM HKD126 has shown that the direction of PLGM affects the directional feature of slope failure.

To identify the directional characteristic of the collapsed slope with the PLGM, the largest pulse is extracted from the velocity time history and computed on the symmetrical slope model using DDA. The reversed wave of extracted PLGM is also obtained for directional characteristic analysis. Figure 5.16 shows the simulated result of the symmetrical slope under the extracted velocity pulse from station HKD128 and the same result with the symmetrical slope subjected to the original PLGM. The west side of the slope collapses under the extracted original velocity pulse while the east side of the slope slides down under the reserved velocity pulse. The collapsed slope is consistent with the direction of the pulse (DOP).

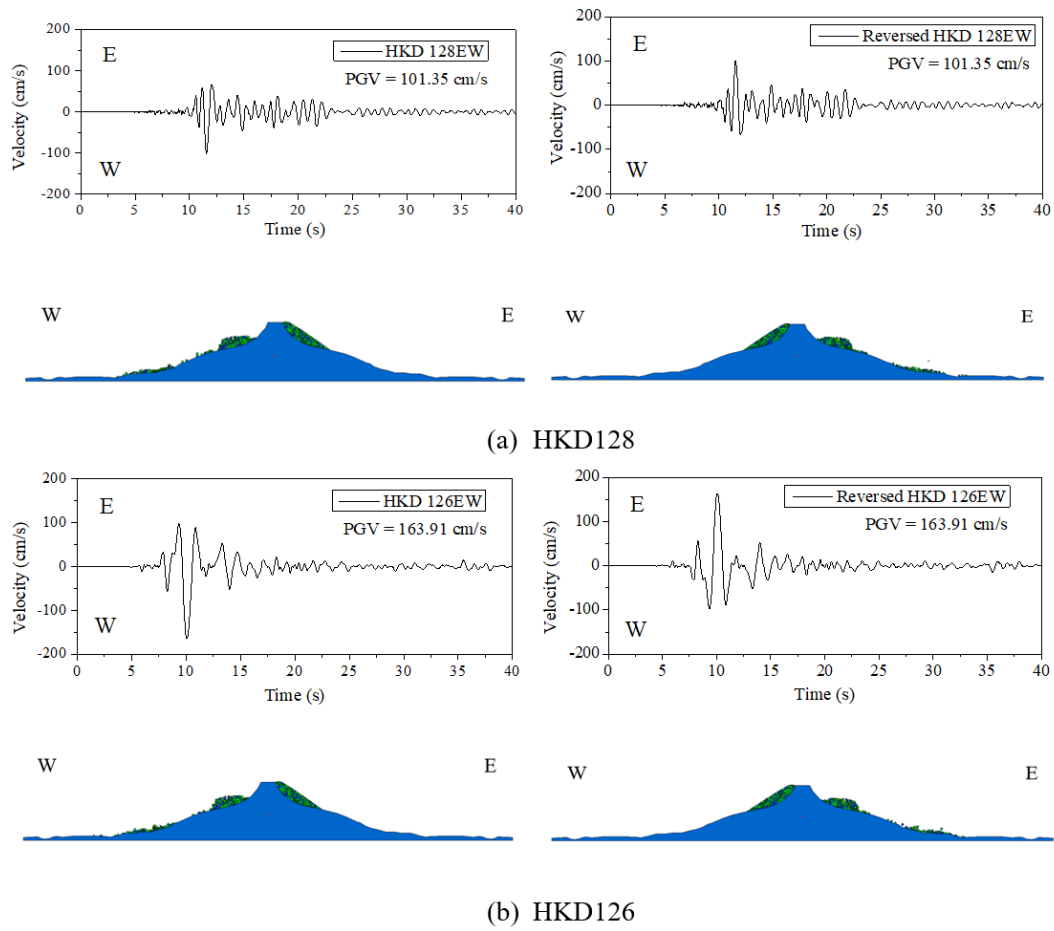


Figure 5.15 Simulation of symmetrical slope under the original and reversed value of PLGM

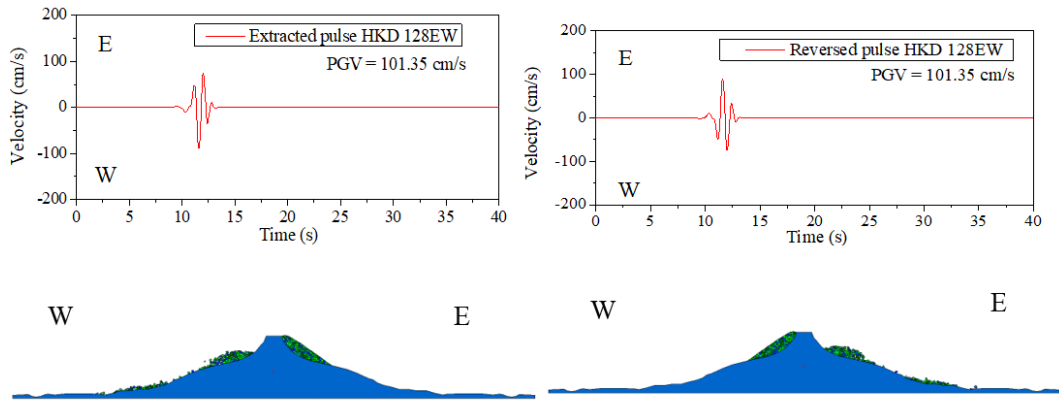


Figure 5.16 Simulation of symmetrical slope under the extracted velocity pulse from station HKD128

To analyze the directional feature of slope failure focusing on energy input, the energy of velocity pulse is then under scrutiny to explain the directional feature of slope failure under PLGM as in the 2018 Hokkaido earthquake and the concentration of landslide area in the 2016 Kumamoto earthquake. As the seismic wave is of component West and East where its positive velocity directs to the East and negative velocity directs to the West, the pulse energy is divided accordingly as shown in Figure 5.17. As the slope aspects also face West and East, the energy absorbed by each slope aspect is also separated based on the positive and negative velocity, respectively. The cumulative pulse energy from HKD126 shows the negative pulse of extracted PLGM at the west side has substantially large energy, which dominates the main part of the total energy. The slopes subjected to PLGM in the Hokkaido earthquake have larger energy consumption on the west side than those on the east side, which might lead to the only one side slope failure phenomenon in the Hokkaido earthquake that the collapsed slopes are west-faced while east-faced slopes back to back remain stable.

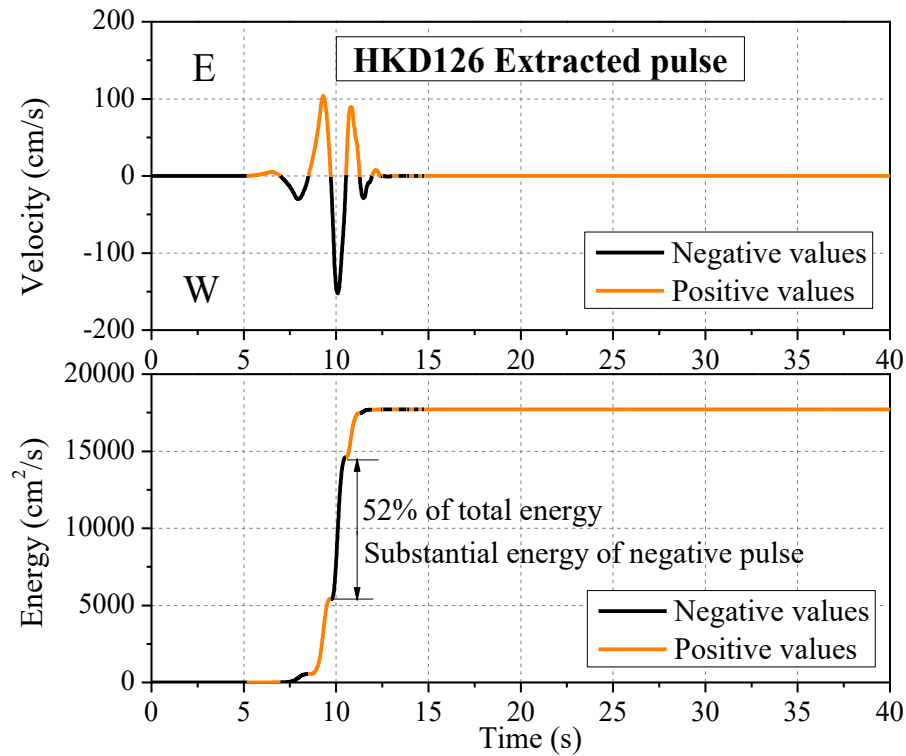


Figure 5.17 Cumulative pulse energy separated by positive and negative values of velocity wave from HKD126 station

5.4.2.2. ASPECT OF THE COLLAPSED SLOPE IS INCONSISTENT WITH THE DIRECTION OF VELOCITY PULSE

By a lot of simulations, we also found that the aspect of the collapsed slope is consistent with either the direction of PLGM in some cases or the opposite direction in some other cases. The collapse direction is related to the physical properties of the slope, that is, the ratio of shearing strength to tensile strength. The slope can collapse at the opposite direction of velocity when the slope can resist the impact of velocity pulse energy but cannot resist the shearing failure. Table 5.5 shows the changes in parameters of symmetrical slope subjected to HKD126. In this situation, the slope failure is shearing controlled and the aspect of the collapsed slope is not consistent with the direction of PLGM. As shown in Figure 5.18, when the symmetrical slope is subjected to the PLGM HK126, the aspect of collapsed slope changes to the opposite direction of PLGM as the tensile strength increasing and shearing strength decreasing. The result is consistent with the simulation result of

the simple model (In Figure 5.19) in which the block slides due to the shearing failure, which proves shearing controlled failure in Fig. 18. In particular, if both the shear strength and tensile strength are very small and not strong enough to resistant the high energy of velocity pulse and shearing force, the collapsed slope will occur at two sides.

Table 5.5 The changes in material and joint parameters of the applied DDA model for HKD126

	Material parameters				Joint parameters		
	Mass (kg/m ³)	Unit weight (N/m ³)	Young's modulus (Pa)	Poisson's ratio	Cohesion (MPa)	Friction angle	Tensile (kPa)
Sliding blocks	2150	-21500	5.4E+9	0.28	3.2 (Decreased)	18.7°	1500 (Increased)
Base block	2.5E+9	0	5.4E+9	0.1			

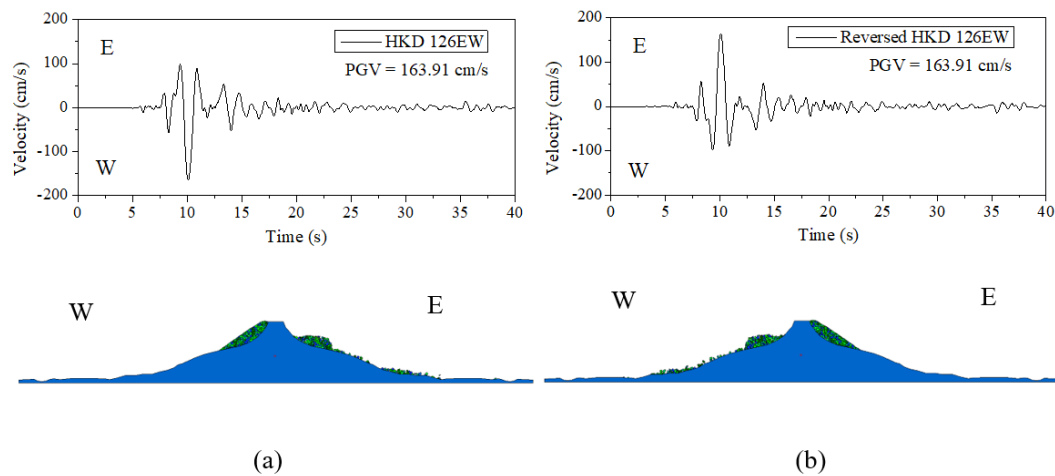


Figure 5.18 Aspect of the collapsed slope is not consistent with the direction of PLGM

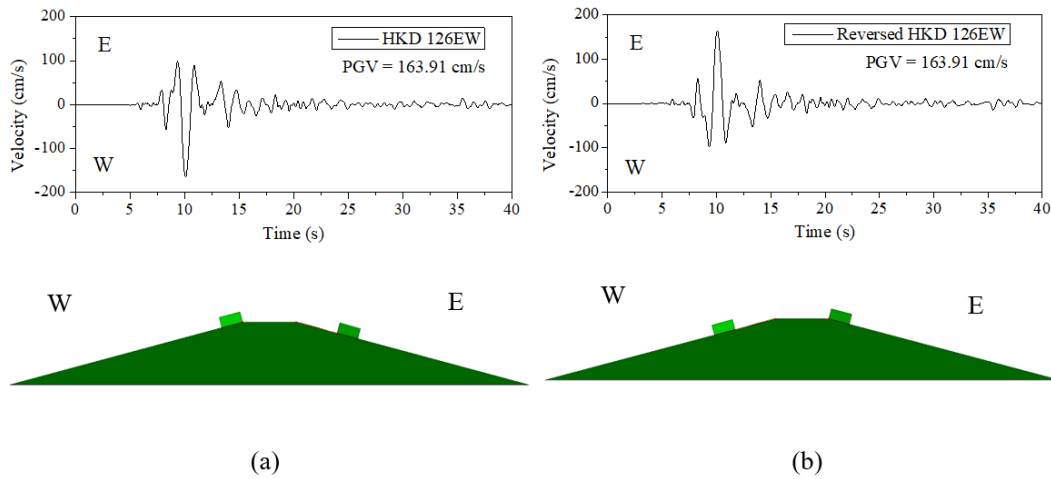


Figure 5.19 Simulation of simple slope under shearing failure

The analysis results show that the directional characteristic of the collapsed slope is not only related to shear strength but also influenced by the tensile strength of slope. The shear failure related to PGA is not the only factor to cause the slope failure, the tensile failure also provided conditions for landslide initiation (Zhao et al., 2019). From this study focusing on the PLGMs in the 2018 Hokkaido earthquake, it can be concluded as two main kinds of slope failures base on both the tensile strength and shear strength: the shear-controlled and the energy-controlled. If the tensile strength of the slope is strong enough and only shearing failure is possible, the slope is prone to collapse at the opposite direction of velocity pulse, the failure of slope behaves as the shear controlled. If the slope has small shear strength and large tensile strength, the slope is prone to collapse in the direction of velocity pulse, the failure of the slope behaves as energy-controlled.

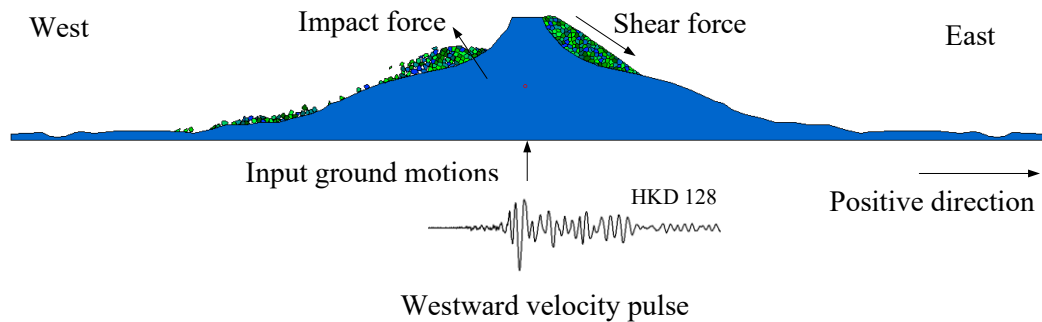


Figure 5.20 Slope failure mechanism under directionally westward velocity pulse of PLGM

As shown in Figure 5.20, the symmetrical slope is subjected to the westward velocity pulse HKD 128. The slope has a large shear force at the opposite direction of velocity pulse due to inertial forces and impact force at the direction of velocity pulse. The simulated results show that the slope collapsed at the east when the slope has large tensile strength and small shear strength. The direction of collapsed slope changes to the west when the tensile strength of the slope decreases, which is consistent with the phenomenon in the 2018 Hokkaido earthquake that the slope collapsed at the direction of velocity pulse under PLGM. If the slope has both small tensile strength and shear strength, the slope is prone to collapse and be discretized in the intense impact, and caused the landslides at the direction of velocity pulse as both tensile and shearing failure occurred simultaneously in this case. In this situation, the failure of the slope has a close relationship with the energy of the velocity pulse. The larger energy the seismic wave has, the larger damage it causes. It indicates that the tensile strength might be the key factor affecting the failure characteristics of the slope in the 2018 Hokkaido earthquake. Field investigation on landslides in the 2018 Hokkaido earthquake shows that the slopes in those areas have the same layers of volcanic matters such as pumice and volcanic ash due to eruptions of volcanoes. These layers have collapsed in the intense earthquake shake and caused multiple landslides (Kazuo et al., 2019). Researches on the properties of volcanic matters show volcanic ash/ pumice has small cohesion but a relatively large friction angle (Shinoda et al., 2019). The shear strength of volcanic matters

could be relatively large but the tensile strength is extremely small. The volcanic ash with low tensile strength is easy to be loose and collapse due to the impact of the westward velocity pulse, which might be the main reason why the slopes mostly collapsed in the direction of the velocity pulse of PLGM in 2018 Hokkaido earthquake. The proposed energy-controlled failure mechanism can well explain the directional characteristic of collapsed slope in the Hokkaido earthquake by using the DDA method.

5.5. DISCUSSIONS

By analyzing the PLGM caused by the forward directivity effect and the large energy it contains, the confusing phenomenon that landslides occurred in the area where PGAs are small while no landslide occurs in nearby mountain areas where PGAs are large can be well explained. By studying the relationship between the direction of velocity and slope's physical properties including tensile and shear strength, the confusing phenomenon that one slope can get failure while the other side slope keeps stable is explained. PLGM is one of the key factors triggering landslides in slope failure analysis due to its substantially large energy. The energy of earthquake seismic waves should be considered in slope stability analysis. Especially in the forward directivity effect area near the seismic fault, PLGM is frequently observed even though it is far from the epicenter. It always has a small PGA but large energy, which is a big threat to slope stability and can cause catastrophic damage. Slope stability analysis focusing on PLGM which contains large seismic energy should be paid more attention in disaster prevention and mitigation project.

However, even in the recently developed method for landslide hazard map, such as the Nowicki model adopted by USGS for evaluating the landslide hazard map, the impact of pulse-like ground motion on landslide initiation is neglected and the seismic energy is not considered for slope stability analysis. PGA is used as one of the key factors triggering landslides, which presents an inconsistent result with the practical landslide initiation. Meanwhile, in numerical methods for slope stability analysis under earthquake, the static shear and tensile strength are mostly

used. For example, the static tensile and shear strength are used in the original DDA method. Once the shear and tensile stresses are larger than the corresponding resistance in the tangential or normal direction, the failure occurs. The effect of seismic energy of PLGM is also neglected. The slope failure in the original DDA is PGA-dependent. How to analyze slope stability under earthquakes by considering seismic energy is a big challenge. Therefore, the proposed method fulfills this scope using the discrete element method for slope stability analysis by considering seismic energy. To effectively analyze the slope failure induced by the earthquake, especially for those PLGM with a small PGA but large energy, the proposed method is recommended. The analysis result in this study has proved that it is effective for slope stability analysis by considering seismic energy. The slope got failed under seismic waves with large energy even though the PGA is small. While the slope remained stable under the seismic waves with small energy even though the PGA is large. The analysis results are in well agreement with the phenomena in those earthquakes with PLGM observed. The analyses in this study provide a means to slope stability analyses by considering seismic energy and PLGM in the forward directivity effect area for future hazard analysis assessment.

5.6. CONCLUSIONS

In this section, a possible mechanism of earthquake-induced landslide based on PLGM has been proposed and discussed. The energy and direction of pulse wavelet in PLGM can be well used to explain the two miserable phenomena (1) why large-scale landslides occurred in the area with small PGA values and (2) failure occurred on the slopes with a specific aspect. An amount of DDA landslide simulations have been carried out and the following results have been obtained:

(1) A PLGM wave can initiate a landslide because it is of large energy. The wave with a large PGA does not always induce a landslide if there is no PLGM contained, which is shown by an example in which the residual wave by removing PLGM from the original wave has a large PGA but cannot initiate landslide. The large-scale landslides induced by the Kumamoto earthquake can be explained very well based on the mechanism. Since the Aso area is in the forward directivity of the

seismic fault, PLGM has been detected in the seismic waves recorded at KMM004, KMM005 and KMM007 around the Aso area. The large energy of PLGMs can destruct slopes although the corresponding PGA is small. On the other hand, no large-scale landslide occurred in other areas with large PGA without PLGM.

(2) For two symmetrical slopes back to back with the same physical and mechanic parameters, one side slope collapses while the other side remains stable. The analysis results show that which side collapses is related to both the direction of the pulse (DOP) and strength parameters of the slope. It can be concluded as two main kinds of slope failures base on both the tensile strength and shear strength: the shear-controlled and the energy-controlled. If both tensile and shearing failure occurred simultaneously, the failure of the slope is regarded as energy-controlled and the aspect of the collapsed slope is consistent with the direction of pulse. If the tensile strength is strong enough and only shearing failure is possible, the failure of slope changes to shearing failure dominated and can be regarded as the shear-controlled, the aspect of collapsed slope could be opposite to the direction of pulse. The phenomenon that many west-faced slopes collapsed while their opposite slopes back to back remain stable during the Ibri earthquake can be explained by the energy-controlled slope failure under PLGM. The volcanic ash with low tensile strength and westward velocity pulse of PLGM are the main reasons for this slope failure phenomenon.

By using the improved DDA method, the slope subjected to PLGM can be well calculated and simulated. DDA has an advantage in simulating the co-seismic landslide initiation because it considers both the shearing failure and tensile failure. The DDA model system is also greatly sensitive to energy analysis which makes it more suitable for co-seismic landslide analysis focusing on PLGM as energy is one of the key parameters to characterize the features of PLGM. The proposed mechanism can be expected in further study to be applied to potential landslide prediction. The quantitative analysis of the relationship between landslide initiation and PLGM will be studied in our future work. Slope stability analysis focusing on PLGM which contains large seismic energy should be paid more attention in disaster prevention and mitigation project. This study provides a method for slope

stability analyses by considering seismic energy. Therefore, it is recommended for slope stability analysis under PLGM in the forward directivity effect area for future hazard analysis assessment.

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CHAPTER 6

CONCLUSIONS AND FUTURE STUDIES

6.1. HIGHLIGHTS AND MAJOR CONCLUSIONS

Earthquake-induced landslide is one of the most destructive secondary geohazards frequently occurred during an earthquake disaster. On the one hand, the triggered landslides always cause serious damage to residents and structures because of their sudden occurrence, high-speed motion, and the large volume of sliding mass. Meanwhile, landslides can further cause a geological disaster chain by forming landslide dams and causing subsequent downstream flooding, or debris flow disaster. The earthquake-induced landslide is still one of the major threats to residents. For example, the 2016 Kumamoto earthquake (M7.3) induced landslides resulting in 15 victims. The 2018 Hokkaido Eastern Iburi earthquake (M6.5) induced landslides resulting in 39 victims but no victim resulted from building collapse due to the advance of quake-resistance technology of buildings. To minimize the damage caused by earthquake disaster, it is necessary and important to clarify the mechanism of the earthquake-induced landslide and develop a more accurate prediction method on landslide-prone slopes based on the active fault information and potential earthquake magnitude.

Over the past decades, many studies on the mechanism of earthquake-induced landslides have been conducted by using pseudo-static methods, Newmark's permanent-displacement methods, and stress-strain methods. The PGA has been

frequently used as an intensity parameter representing the earthquake force in the methods for seismic slope stability analysis, such as pseudo-static methods. However, in recent near-fault earthquake-induced landslides, there are two confusing landslide phenomena that cannot be well explained by using PGA: (1) large-scale landslides occurred in the area with small PGA but no landslide in the area with large PGA; (2) one side slopes collapsed while the opposite side slopes back to back remain stable even though there is no big difference on PGA at two sides. To completely analyze the mechanism of earthquake-induced landslides, studies on landslide initiation focusing on PLGM are conducted by using the DDA method. DDA is a powerful numerical method for landslide simulation and has great advantages in simulating the dynamic behaviors of a rock mass system involving discontinuities. At first, to achieve the analysis on earthquake-induced landslides using DDA in this study, some necessary improvements and extensions are also conducted. Then, studies on the mechanism of earthquake-induced landslides focusing on pulse-like ground motions have been performed by using the improved DDA.

Firstly, the vertex-to-vertex contact processing in 2D DDA has been improved by a new concept of the vertex-to-vertex contact detection algorithm. In the original DDA, a case of vertex-to-vertex contact state was not considered, which will lead to indeterminacy in seismic response analysis. To overcome this defect, a new improved concept based on the contact vector method that is inside or outside the reference line has been proposed and implemented in the 2D-DDA program. The accuracy and effectiveness of the new vertex-to-vertex contact detection algorithm have also been verified by several contact simulation examples, including the lost case contact, sliding block contact, blocks contact with convex, and blocks contact with concave. The simulated results have proved that the lost case of contact detection in O-DDA has been well solved. The I-DDA also has better performance and accuracy in solving block contact problems in various cases. Therefore, the I-DDA with a new vertex-to-vertex contact detection algorithm can be better applied to block systems with arbitrary vertex-to-vertex contact states.

Then, the tensile failure model in 2D DDA has been extended for simulating the rock fracture by incorporating a unified tensile fracture criterion. The new tensile fracture criterion unifies four classical failure modes, including the maximum normal stress criterion, Tresca criterion, Mohr-Coulomb criterion, and von Mises criterion, which can be well applied to analyze rock fracture problem of various materials. The accuracy and effectiveness of the I-DDA method have been validated by numerically simulating crack initiation and propagation in several Brazil disc split tests. The simulated results agree well with those from experiment tests. Meanwhile, an earthquake-induced landslide event has been analyzed using the I-DDA method with both the tensile and shear strengths considered. The simulation results show the I-DDA with a unified tensile fracture model can also be effectively applied to landslide simulation. Different failure behavior considering both tensile and shear strengths have been concluded by using the I-DDA method.

Thirdly, a dynamic failure method considering the energy of seismic waves has been proposed for evaluating slope failure under earthquakes in DDA. This method can be applied to effectively evaluate the dynamic strength of joint contacts in the DDA method by considering the effect of fatigue and accumulated damage under seismic dynamic loading. The Single-Degree-Of-Freedom (SDOF) systems under cyclic loading, PLGM, and Non-PLGM has been simulated by using the proposed DDA method and original DDA to validate the accuracy of the proposed model in describing the dynamic failure of contact joints. Then, a practical landslide model has been simulated under two cases of seismic waves: case (A) PLGM with large energy but a small PGA; and case (B) Non-PLGM with a large PGA but small energy, using both the proposed DDA method and O-DDA. The results showed that the slope gets failure by using the proposed method while keeps stable by using the original DDA for Case A. For Case B, the reversed results have been obtained. The accuracy and effectiveness of the proposed method are validated and the energy of seismic waves is a key triggering factor in slope stability analysis under earthquakes. Compared with O-DDA, the proposed DDA method is effective for simulating the earthquake-induced landslides considering the seismic energy, which is more accurate for earthquake-induced landslides triggered by PLGM.

Finally, a study on the mechanism of earthquake-induced landslides focusing on PLGM has been conducted by using the improved DDA method. To analyze the two confusing landslide phenomena in the 2016 Kumamoto earthquake and the 2018 Hokkaido earthquake, the mechanism of earthquake-induced landslides are investigated focusing on PLGM. Firstly, the PLGM has been introduced and PLGMs in historical earthquakes in Japan have been identified by analyzing the recorded ground motions from K-NET and KiK-net databases. Then, the phenomenon (1) was investigated by DDA simulation on Aso-Bridge landslide in the 2016 Kumamoto earthquake under PLGM with a small PGA and Non-PLGM with a large PGA. The phenomenon (2) was studied by simulating the symmetrical DDA model considering the pulse direction and slope's physical properties including tensile and shear strengths. The results shows that which side collapses is related to both the direction of the pulse (DOP) and strength parameters of the slope. It can be concluded as two main kinds of slope failures base on both the tensile strength and shear strength: the shear-controlled and the energy-controlled. If both tensile and shearing failure occurred simultaneously, the failure of the slope is regarded as energy-controlled and the aspect of the collapsed slope is consistent with the direction of pulse. If the tensile strength is strong enough and only shearing failure is possible, the failure of slope changes to shearing failure dominated and can be regarded as the shear-controlled, the aspect of collapsed slope could be opposite to the direction of pulse. Finally, a possible mechanism of earthquake-induced landslides focusing on pulse-like ground motions has been proposed. The two confusing landslide phenomena can be well explained by the proposed mechanism of earthquake-induced landslides focusing on PLGMs.

6.2. FUTURE STUDIES

This study aims to analyze the mechanism of earthquake-induced landslides focusing on the PLGM by using Discontinuous Deformation Analysis (DDA). To achieve the analysis of earthquake-induced landslides with the PLGM by using the DDA method, some improvements and extensions have been conducted firstly for better calculation performance and applicability. Then, a dynamic failure method

considering the energy of seismic waves for slope stability under earthquakes has been proposed in DDA simulation. With the improved DDA, the study on the mechanism of earthquake-induced landslides focusing on the PLGM have been conducted by considering the velocity pulse and slope's physical properties to clarify some confusing landslide phenomena in earthquake events. Therefore, a possible mechanism of earthquake-induced landslides focusing on pulse-like ground motions has been proposed. However, there is still space for further improvements on the following aspects.

(1) The I-DDA with the unified tensile fracture model (UTFM) has been proven to be better applied to analyze the tensile behavior of various materials. Different slope failure models with different tensile strengths, including tensile-controlled, shear-controlled and tensile-influenced, shear-controlled can be simulated by using the I-DDA method. The slope stability has been proved to be significantly affected by both the tensile and shear strength under seismic conditions. However, slope stability analysis mostly focuses on the shear failure and shear characteristic of landslide mass in most case studies. More studies focusing on both shear and tensile failure should be considered in future work.

(2) A method considering the dynamic strength of slope material has been proposed for earthquake-induced landslide in DDA simulation. This method evaluates the dynamic failure of joint contacts in the DDA by considering the effect of fatigue and accumulated damage under seismic dynamic loading. The effectiveness of the proposed model considering the dynamic strength failure has been verified by numerical simulations, how to take the parameters for the dynamic model under seismic waves has not been considered in this study. For quantitatively evaluating the initiation of earthquake-induced landslide by using the proposed DDA method, more experimental studies on the dynamic failure strengths of rock mass about the parameters α , N_f and R_{res} should be performed in future work. The quantitative relationship between the energy of PLGM and landslide initiation should be studied to quantitatively analyze the mechanism of earthquake-induced landslides.

(3) A possible mechanism of earthquake-induced landslides focusing on pulse-like ground motions has been proposed. But this study has been mainly conducted by numerical simulations with the DDA method based on the landslide cases in the 2016 Kumamoto earthquake and the 2018 Hokkaido earthquake. More experimental studies on the physical slope model and case studies with PLGM should be conducted for a more accurate evaluation of the mechanism of earthquake-induced landslides under PLGM.

(4) Earthquake-induced landslide is a complex geological hazard problem. The initiation of an earthquake-induced landslide is related to both the slope's inherent factors such as geological and physical properties and the external earthquake triggering force. The study on the mechanism of the earthquake-induced landslide has been performed by considering the velocity pulse and slope's tensile and shear strengths. The other factors such as geological properties and slope geometry should also be considered for accurate estimations of an earthquake-induced landslide in the future.